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Sewerage and Irrigation

97 ILLUSTRATIONS

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SEWERAGE
SEWAGE TREATMENT
IRRIGATION

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PREFACE

The volumes of the International Library of Technology are made up of Instruction Papers, or Sections, comprising the various courses of instruction for students of the International Correspondence Schools. The original manuscripts are prepared by persons thoroughly qualified both technically and by experience to write with authority, and in many cases they are regularly employed elsewhere in practical work as experts. The manuscripts are then carefully edited to make them suitable for correspondence instruction. The Instruction Papers are written clearly and in the simplest language possible, so as to make them readily understood by all students. Necessary technical expressions are clearly explained when introduced.

The great majority of our students wish to prepare themselves for advancement in their vocations or to qualify for more congenial occupations. Usually they are employed and able to devote only a few hours a day to study. Therefore every effort must be made to give them practical and accurate information in clear and concise form and to make this information include all of the essentials but none of the non-essentials. To make the text clear, illustrations are used freely. These illustrations are especially made by our own Illustrating Department in order to adapt them fully to the requirements of the text.

In the table of contents that immediately follows are given the titles of the Sections included in this volume, and under each title are listed the main topics discussed.

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SEWERAGE
(PART 1)

SEWERAGE SYSTEMS

PRELIMINARY INFORMATION

1. Sewage and Sewerage.—In connection with the contamination of water supplies, *sewage* is defined as the refuse or excreta from animals, including human beings. In its more general meaning, however, the term *sewage* is used to denote all the liquid wastes of a city, town, village, or other community. It consists of a comparatively large amount of dirty water and a relatively small quantity of solids, which include human excrement from flush toilets, soap from baths, grease and small particles of garbage from kitchens, and other wastes. Although, in all, these solids make up only 1 or 2 parts to a thousand parts of water, they are sufficient to make the entire volume offensive, for they are dangerous to health and are liable to form deposits causing odor and nuisance. In some cities, factories and shops add manufacturing wastes to the sewage; in other places, the run-off from rains, or *storm water*, is carried away together with the other wastes.

The removal of the sewage is ordinarily accomplished by a system of pipes or drains, called *sewers*. The process of removal is designated as *sewerage*, and the system of pipes and drains is called a *sewerage system*.

2. Functions of Sewerage Systems.—A good sewerage system should provide for the prompt removal of all domestic.

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or house and manufacturing, sewage, and of all storm or waste water from the ground surface or the immediate subsoil. These wastes should be conducted rapidly and without nuisance or danger to health to some place where treatment or final disposal takes place. Sewers have two main functions: health protection and convenience. The primary requisite of health protection is too often lost sight of, and the sewer viewed only as a convenient necessity of modern life.

3. Kinds of Sewerage Systems.—From the engineering standpoint, the subjects of drainage and sewerage are intimately connected. In general, however, sewerage refers principally to the removal of human refuse, excreta, and other waste matter common to communities, whereas drainage relates to the removal of storm or ground water from the surface or subsoil. All water produced by rain-storms may be called storm water, but this term is commonly applied to that water from rain-storms that neither soaks into the soil immediately nor evaporates, but flows away over the surface through natural channels, or is carried away by artificial conduits.

Sewers may be designed to carry storm water alone, house refuse alone, or both together. Those that carry storm water alone are called *storm sewers;* those that carry house sewage alone are called *house sewers.* The system involving storm water sewers is called the *storm-water system,* and that involving house sewers is called the *separate system,* to indicate that the house sewage is separated from the rain water. Some cities use the *combined system,* through the pipes of which both storm water and house sewage are removed.

4. Requirements of Sewerage Systems.—Health and sanitation demand complete and immediate removal of all closet and house wastes, and a separate system should be designed to carry off these wastes promptly, without odors, and without interruption. Moreover, the sewer should keep itself clean, that is, free from deposits; it should not pollute the soil through which it passes; and it should have an outlet that is not objectionable.

A storm-water system should be adequate for the prompt removal, from the ground surface, of such storm water as may be produced by a violent storm, including also such refuse from the streets as may ordinarily or necessarily be carried away by the storm water. In addition, since sewers are rarely tight, some water will enter the sewer from the subsoil.

5. Relative Importance of House and Storm Sewers. The house sewers remove the really foul and dangerous waste products of the community, whereas the storm sewers remove mainly the water resulting from rainfall. Moreover the necessity for the removal of house sewage is continuous, while storm sewers are required only at intervals. Furthermore, leaky and overflowing house sewers cause conditions dangerous to health, but the inconvenience of flooded gutters can be withstood.

For the purpose of illustrating the relative capacities that must be provided to care for storm water and for house sewage, the following rough calculation may be made: Assume that the population on an area of 1 square mile is 10,000; that each person contributes 75 gallons of house sewage per day of 16 hours; and that the capacity of the storm sewers must be sufficient to remove 1 inch of rainfall per hour from the entire area, which is a fair allowance for this density of population under ordinary conditions. Then since the population per acre is $\frac{10,000}{640}$ and the rate of sewage discharge per person is $\frac{75}{16\times 7.48}$ cubic feet per hour, the house sewage will be discharged at the rate of $\frac{10,000}{640}\times\frac{75}{16\times 7.48}$, or 10, cubic feet per acre per hour; and since the volume of a prism whose base has an area of 1 acre, or 43,560 square feet, and whose altitude is 1 inch, or $\frac{1}{12}$ foot, is $43,560\times\frac{1}{12}=3,630$ cubic feet, storm water will be discharged at the rate of about 3,600 cubic feet per acre per hour. The discharges and, consequently, the relative capacities of the required sewers, will be in the ratio of about 1 to 360. Therefore, the capacity of a combined sewer need not be much greater than that of a storm-water sewer.

6. Comparison of Separate and Combined Systems. Where both house sewage and storm water must be removed, it is necessary to choose between a combined system, on the one hand, and storm-water and separate systems, on the other. The difference in capacity between a combined sewer and a storm-water sewer, as shown in the preceding article, is negligible. It is likely to cost more, therefore, and requires more room in the street, to construct two systems, one carrying storm water and the other house sewage, than to build a combined system. Nevertheless, for most conditions a separate system is preferable, as it possesses several important advantages.

The separate system, if properly constructed, will more thoroughly meet the requirements for the efficient removal of house sewage, and will be more strictly sanitary than the combined system. In the separate system, the volume of sewage and the velocity will be comparatively constant, in dry as well as in wet weather, and there will be little tendency for permanent deposits to collect. For this reason, and also on account of their freedom from street detritus, such as sand and gravel, separate sewers generally can be constructed with flatter slopes than would be required in combined sewers. When flushing is necessary, the same degree of cleanliness can be attained with less water in the separate than in the combined system because the sections are smaller. Where the sewage has to be treated, the use of the separate system is practically a necessity, for the treatment, handling, and pumping of all the storm sewage is almost impossible and very costly.

7. Extent of Use of Separate System.—It is to the considerations just stated that the popularity of the separate system, and its adoption in so many cities, are due. Whether a separate system should be adopted for all cities is questionable, but generally it is advantageous. Few cities have built combined sewers during the twentieth century except in the case of extensions to existing combined sewers.

ARRANGEMENT OF SEWERAGE SYSTEMS

8. Controlling Factors.—The topography of the city and the method of treating and disposing of the sewage will generally be the deciding factors in the selection of a sewerage plan. The routes of the main sewers should, in a general way, follow the natural drainage channels, for the best grades will usually be obtained by this plan; but the entire system cannot always be laid out to conform to the topography. If this method were applied to a city that lies partly on a hillside and partly on a level plain, the sewer grades in the higher parts of the city would be so steep that, in times of storms, the sewers on the lower levels would have to be excessively large to carry off the water without damage. The best arrangement in such a case is to divide the city into districts, the sewerage systems for the districts being connected.

The kind of treatment to be given the sewage may have an important bearing on the general design of a sewerage system. Where disposal is directly into a large body of water, on the shores of which the city is located, the effect on the design will be small. However, if it is objectionable to discharge the sewage close to the city, provision must be made to carry it, before discharge, a sufficient distance down stream to eliminate offensive conditions. When sewage is treated, additional complications arise, as a site for a sewage treatment plant is not always easy to find. It may be necessary to transport the sewage for a considerable distance to avoid locating the plant in a thickly settled area, where it might be objectionable or where space might be difficult to obtain. Besides, a certain fall is required through the treatment works, the amount of which varies with the method of treatment.

All these, and many other, factors have to be considered in planning and designing a sewerage system. Each city will present its own problems, and no set plan can be rigidly followed, but certain typical methods of dividing the territory and laying out sewers may be recognized by their features. The more common plans are the *perpendicular, intercepting, fan, zone,* and *radial.*

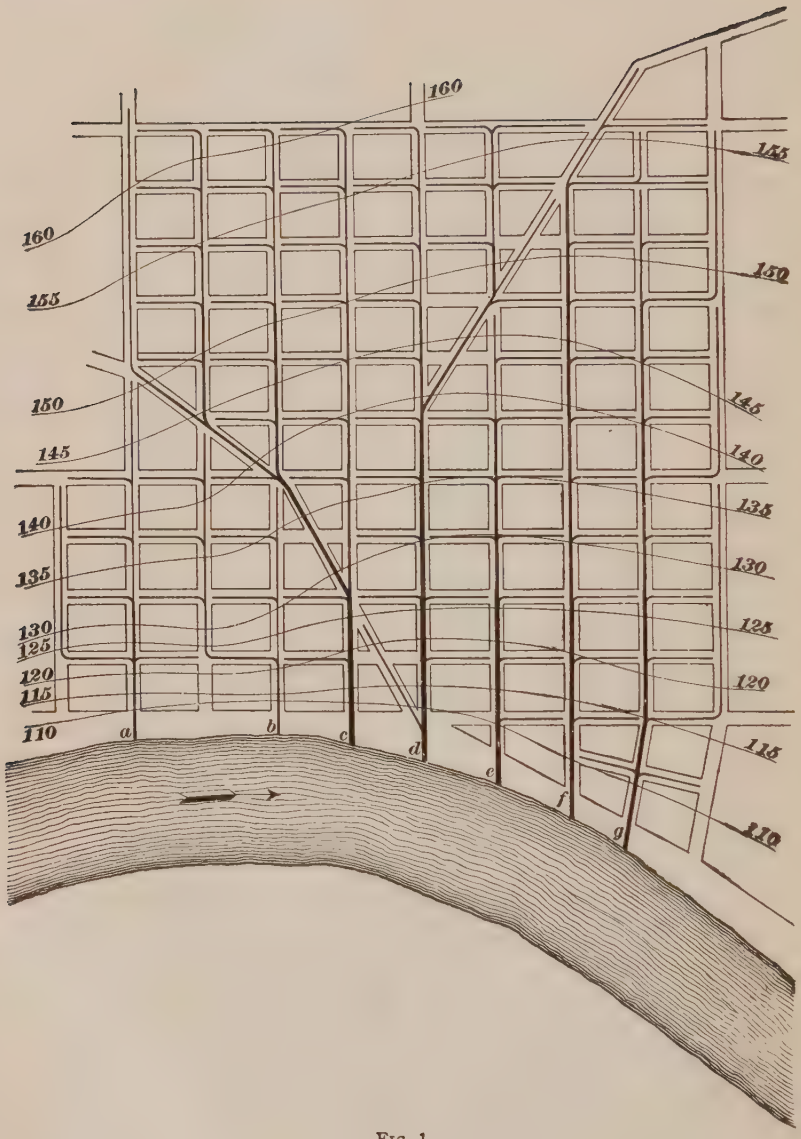
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Fig. 1

9. Kinds of Sewers.—In all sewerage systems, there are main sewers, and branches that discharge into the mains. A sewer that collects the flow from the houses or from the streets, but does not receive sewage from any other sewer, is called a *lateral sewer;* a sewer into which two or more laterals discharge, is a *submain;* and a sewer that receives the flow from two or more submains is a *main sewer* or *trunk sewer.*

When the sewage cannot be discharged directly into a body of water, the mains of a separate or a combined system discharge into a sewer constructed along the bank of the stream or other body of water. Such a sewer is called an *intercepting sewer.* A sewer that carries sewage from the lower end of a collecting system to a suitable point of discharge is an *outfall sewer.*

10. Perpendicular Plan.—When a city is bounded on one or more sides by a body of water, or divided by a stream flowing through it, the plan will generally call for a number of districts with entirely distinct sewerage systems, each having a main sewer with a direction approximately perpendicular to the body of water into which it discharges. A complete perpendicular plan is shown in Fig. 1. There is a sewer in almost every block, and the flow in each case follows roughly the natural drainage channels. The sewers *a, b, c, d, e, f,* and *g* discharge into the stream. Usually, the sewers in a town are not so close together as shown here.

A perpendicular plan has the advantage that the sewers in any locality to which it is adaptable are as short and as small in cross-section as possible. It is generally not only the cheapest, but the most convenient plan, and the one usually followed by a town before any complete system is adopted. Where separate sewers discharge into a body of water in this way, however, there is the disadvantage that the water becomes contaminated within the limits of the town. It may, therefore, be best to collect the sewage from separate systems or the dry-weather flow from combined sewers, and discharge it beyond the city limits at some point that is considered to be safe. The storm flow may usually be discharged directly into the stream.

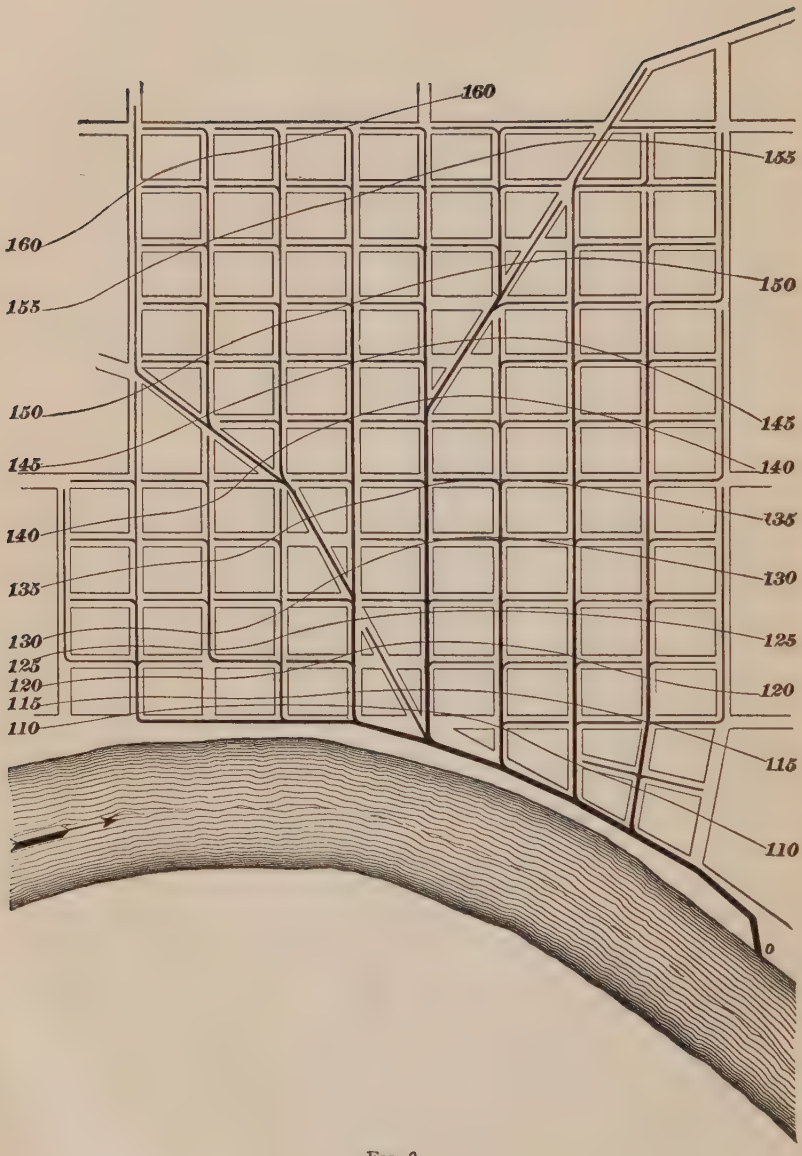

Fig. 2

11. Intercepting Plan.—Contamination of a stream within the city limits may be eliminated by constructing an intercepting sewer and, if necessary, an outfall sewer to convey the objectionable sewage to a treatment plant or to some point suitable for discharge. An intercepting system is, therefore, simply a modification of the perpendicular system. In Fig. 2 is shown an intercepting plan for the same town as is represented in Fig. 1. The intercepting sewer discharges all the sewage at *o*. Although an intercepting plan is not always best when the entire sewerage system can be constructed new, it is of especial value in remodeling a perpendicular system and in adapting old construction to new purposes.

12. Fan Plan.—When the ground surface slopes in such a manner as to form a basin, the sewage from an entire district or city may be conveyed to a single point by means of a number of converging trunk sewers and their branches. The sewers are then said to be laid out according to the fan plan. This system is somewhat preferable to the intercepting system for many cities. In Fig. 3 is shown the same street layout as is represented in Figs. 1 and 2, but the slope of the surface is changed somewhat, as indicated by the contours. In this illustration, the fan plan is applied; most of the sewage is carried to the point *a*, and all of it is discharged at *o*, the sewer between *a* and *o* serving as an intercepting, as well as an outfall, sewer.

13. Zone Plan.—The zone plan is especially adaptable to a city in which the surface consists of a series of shelves or steps differing considerably in elevation. In this case, each shelf, or zone, may have its own sewerage system, which is laid out according to either the fan or the intercepting plan. The street layout represented in Figs. 1, 2, and 3 is again shown in Fig. 4, but the surface elevations are different. In this figure, the zone system is illustrated, the outlet for the entire town being at *o*. An advantage of the zone system is that the sewage and storm water is removed from each level through its own sewers, and, therefore, the troubles and dangers of flooding the lower areas of the city are greatly reduced.

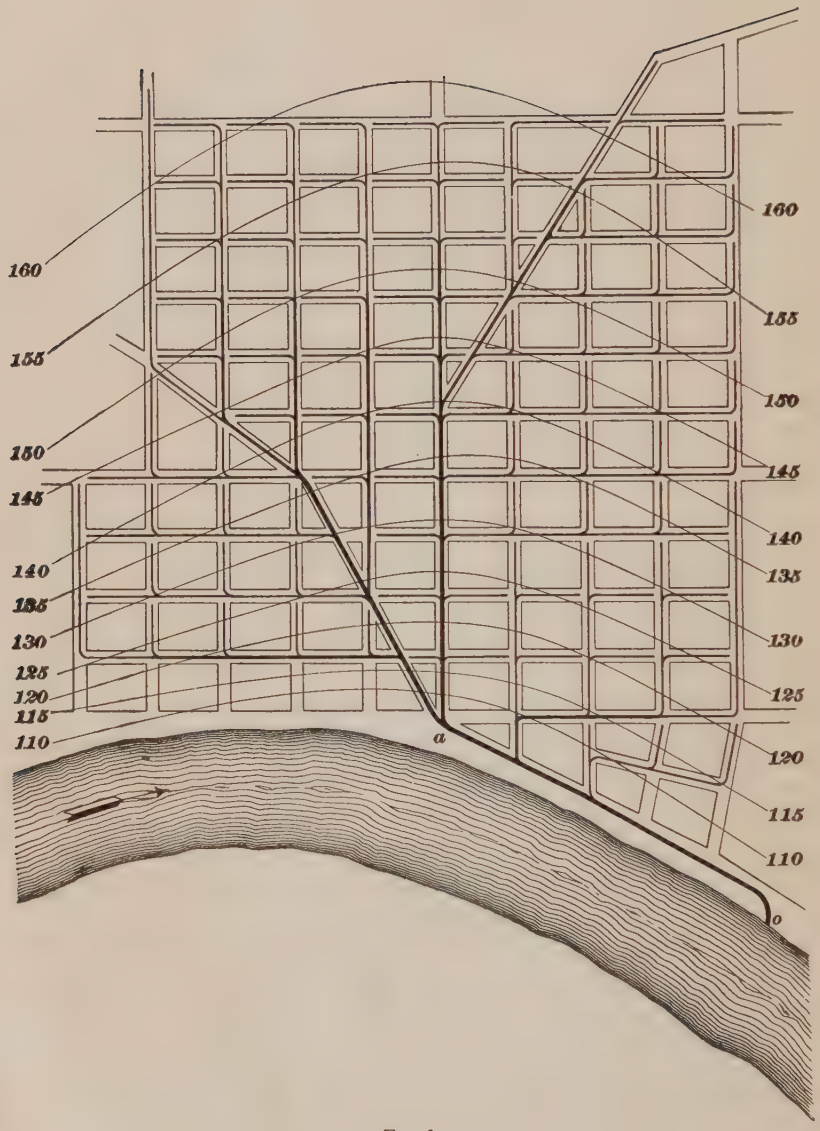

Fig. 3

14. Radial Plan.—In a city where the greatest elevation is in the central part and the ground slopes away in all direc-

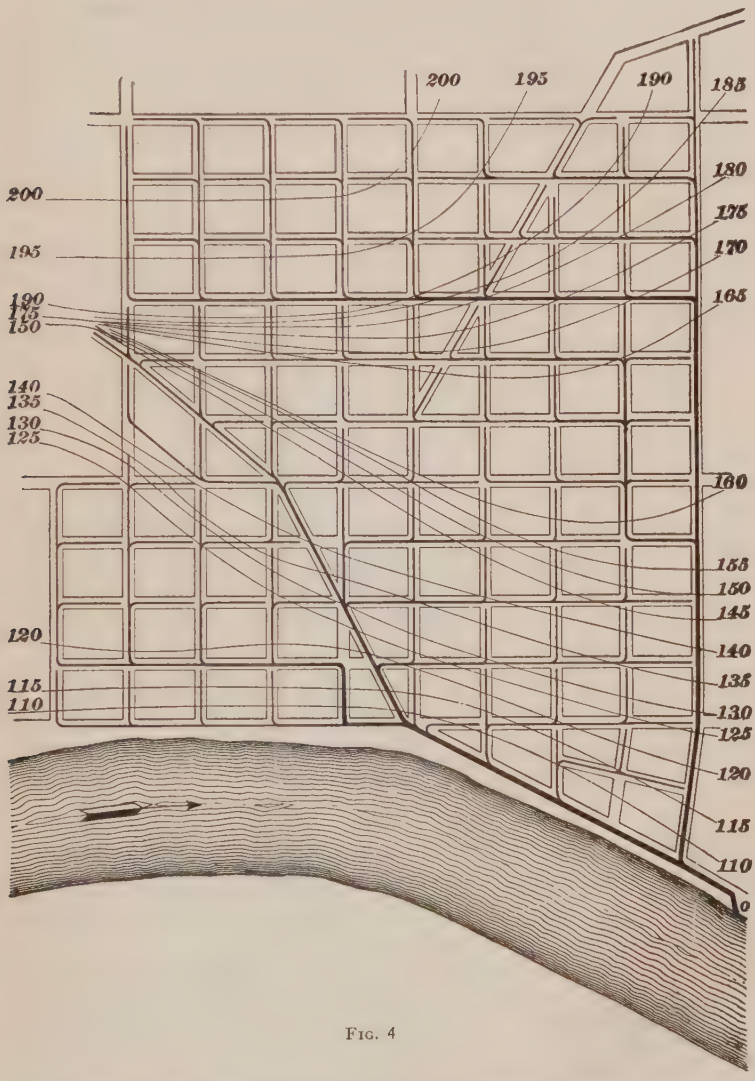

Fig. 4

tions, the radial plan of laying out the sewers is very effective. In applying this plan, the city is divided into a number of

districts corresponding somewhat to sectors of a circle, and the sewage from each district is carried outwards from the center. One great advantage of this plan is that all the sewers in the center of the city are small, and the size of a sewer increases as its distance from the center of the city becomes greater. Moreover, the capacity of the sewers will be sufficient for a very long time, because, as the city grows, the requirements are generally met by simply extending the existing sewers with enlarged sections.

15. Remarks on Different Plans.—In very many cases, the requirements will best be fulfilled by a combination of two or more of the foregoing plans. In general, it may be stated that the greatest degree of economy is obtained by concentrating the systems as much as possible, for a few reasonably large sewers can be constructed more cheaply than a large number of small sewers having the same capacity.

It is probable that, if all considerations of future requirements are neglected, the fan plan is, for the greatest number of cases, the most economical. The intercepting plan can usually be made to serve most economically in cities that are partly sewered, and is commonly preferable when the sewage is to be collected for treatment. For cities in which the surface slopes outwards from the center toward lower points that afford suitable outlets, the radial plan is generally best.

Special conditions require complicated and expensive sewerage systems. If a district is situated in a valley whose general direction diverges from the stream into which the sewage is to be discharged, circuitous courses, as well as deep excavations, are necessary. Such a case is represented in Fig. 5, where a crest of elevated ground, indicated by the dotted line, separates the town into two drainage districts. The outfall for the district *a b* is at *f*, while the sewage from *c* passes around by the long course *d c* and is discharged at *g*; the sewage from *c* cannot be carried directly to the outlet without an exceedingly deep excavation through the high ground.

If a drainage district is nearly flat and at a very slight elevation above the river into which the sewage is to be discharged,

the outfall sewer must be constructed to the lowest available point of discharge. Even then, it may be necessary to resort to pumping in order to provide sufficient fall for the sewage to flow to the outlet. In such a case, the sewers empty into a tank located at the lowest point in the district, and the tank

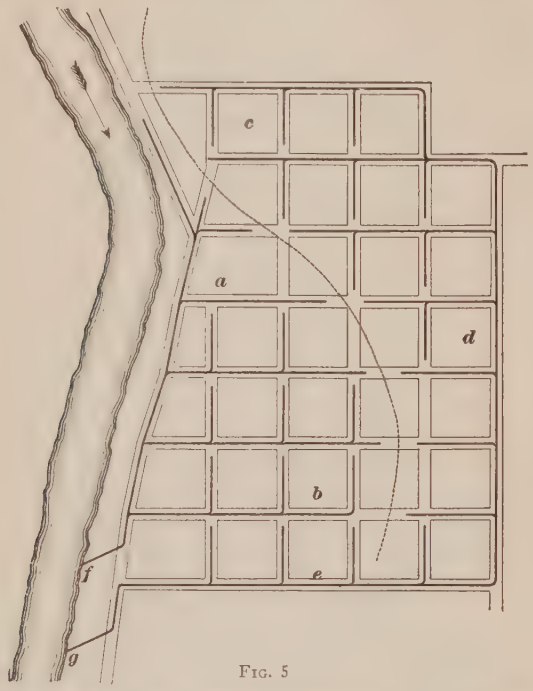

FIG. 5

contents are pumped to a height sufficient to cause them to flow to the outlet by gravity.

16. Capacity for Future Growth.—The limits of a city may be extended indefinitely, and the volume of sewage from the extreme districts correspondingly increased. Since in each of the sewerage systems previously described, except the radial, the sewage must be conveyed from the outskirts toward the center of the city, the trunk sewers either must be designed with a capacity sufficient to take care of future requirements, or must be enlarged from time to time to provide for the sew-

age from new branches that are added as the city grows. If a main is designed for future needs, its cost will be much greater than if present purposes alone are considered. On the other hand, when the sewers must be enlarged the cost of construction will be very high, as the new sewers are trunk lines and must be laid near the center of the city. It should always be kept in mind that the future requirements cannot be estimated accurately, and the growth of the city may be so rapid as to necessitate new sewers within a short time, in spite of the attempt to avoid this extra work. From the information gathered in the preliminary investigations, and from his past experience, the engineer must decide just how far the plans should provide for the future.

SURVEYS AND MAPS

17. Preliminary Investigations.—Before the formulation of a plan for any system of sewers is undertaken, the entire territory under consideration, and also such adjoining areas as reasonably may be expected to form a part of the same system at some future time, should be studied carefully. Surveys and levels should be run; the character of the population, and the kinds of industries and manufacturing plants should be noted; information regarding the nature of the surface of the ground, the type of subsoil, as rock or quicksand, the depths of cellars and basements, and the depth of ground water should be obtained. The present and prospective water supply and population should be investigated, and the rate of local rainfall during a storm estimated. The velocity and volume of the flow in streams into which sewage may be discharged must be determined, as these factors often affect the location of the point of discharge.

18. Surveys and Levels.—Usually surveys and maps of the territory, already made for other purposes, are available for use in the preliminary sewer investigations, or at least sewer maps may be prepared from them without entirely new and independent surveys; but unless these maps are known to be

accurate, check measurements should be taken to determine their value. It is generally necessary to run new and independent lines of levels over the whole area, for it is usually a waste of time to attempt to use recorded profiles of street grades. It is also necessary, in most cases, to make supplementary surveys for outlets and for main sewers along lines not coinciding with street lines. Since extreme accuracy in the transit lines is not a necessity, a stadia survey will be of value, as it gives a maximum of information for a small amount of work. It is desirable to make the preliminary surveys and to keep the records of them in such a way that they will be of greatest service in construction. It is also very desirable to use the same bench marks in the final construction as are used in taking the preliminary levels.

19. Datum Plane and Controlling Bench Mark.—Before the levels are taken, a datum plane should be assumed to which all elevations should be referred. The datum should be lower than the lowest point in the proposed sewerage system, and some controlling bench mark should be assumed as nearly as possible in the center of the area under consideration. Often it is convenient to make the bench mark some substantial mark or point on a public building or structure. If a suitable datum plane has been established for other surveys in the city, the same datum should be used for sewer work. This will make it possible to use much of the information that has been gained in other surveys without recomputing the elevations, and the levels that are taken for the sewer work can also be utilized conveniently for other work.

20. Auxiliary Bench Marks.—After the datum plane and the controlling bench mark have been chosen, a system of bench marks should be established at short intervals over the entire territory, preferably not over 1,000 feet apart. This work should be done carefully, readings on turning points and bench marks being taken to thousandths of a foot. It should be done separately rather than at the same time that surface levels are taken. The lines of levels run in establishing these bench marks should form complete circuits and should be run

in both directions, readings being taken on all bench marks in each direction. If the elevations that are thus found for each bench mark are reasonably close, they may be averaged. A table of bench-mark elevations should then be made out for use, not only in the preliminary levels, but also in the final location and construction of the sewers.

21. Surface Levels.—After the elevations of the bench marks have been determined, the surface levels at the center line and at each side of the street should be taken. The line of levels should start from an established bench mark, check on all bench marks that are passed, and close on a bench mark. Surface-level readings need be taken only to the nearest tenth of a foot. In all cases where a check reading on a bench mark is taken, the field notes should show the error in closing. These closing measurements are often of the greatest value in locating large errors, which may be corrected at once. A small error in closing on an established bench mark should be dropped, and the work resumed from that bench mark as a basis.

It is well to note the elevations at each side of the street, because these really determine the elevation of the sewer; the elevation of the center of the street is of importance only in determining the depth of cut after the proper elevation of the sewer has been fixed. The distance back from the street line to the buildings or basements, the elevations of the basement floors that have already been built, and, if it is probable that the territory will further develop, the depths of the basements that may be expected in future buildings, should be noted. The elevation of a basement will best be determined by taking a reading with the leveling rod on the main-floor line of the building as the level party is passing along the street and, at some later time, taking the measurement from the main-floor line to the bottom of the basement; by following this plan, the progress of the level party will not be delayed.

The preliminary surface surveys are not usually those on which the final profiles and estimates are based. It is not generally practicable at this stage of the work to determine the precise location of the sewers in the streets, and they may be

finally located at one side of the line on which the levels are run. The final surface profiles are made after a line has been adopted and actually staked out on the ground.

22. Special Surveys and Investigations.—The topography of the city is often such that some of the main sewers cannot well follow the streets entirely, but must be run along valleys which form the natural drainage channels for the surface water and which are also the best places for connecting the branch sewers. This is the case where a deep valley intersects the streets in an irregular manner, so that the branch sewers cannot be brought readily across it, but must descend into it abruptly from either side, and must be joined to a main sewer following the natural depression. Special surveys and preliminary location of the line are generally necessary in such cases. Sometimes bridge abutments and other obstructions are factors in determining the most feasible route for the main sewer. The cost of sewers is materially greater when they are built in unstable ground and in wet trenches, and therefore the character of the soil and subsoil should be investigated thoroughly.

The locations of the outlets often become apparent on a superficial examination, and generally are determined before the preliminary work has progressed very far. If the outlets are to be in streams, the condition of the channel, the currents at all stages of water, and the difficulties that may be experienced during freshets, or when ice is running, should be noted. In tidal waters, studies of the currents and water movements are necessary in order to locate the outlets so as to avoid possibility of nuisance and to insure thorough mixing of the sewage with the water.

In all streets there are water and gas mains and other conduits of various kinds which must be avoided in locating the sewers. As they often lie in such positions that they must be considered in determining the grade of the sewers crossing them, their positions and elevations should be determined and recorded.

23. Construction of Maps.—With the surveys and levels at hand, and with such maps as may be available for a basis, a

topographic map of the city should be made. A good scale to use is 200 feet to the inch, but where this scale would make a map too large to use easily, 400 feet to the inch may be used. In some cities it will be convenient to have a separate map for each drainage area. For undulating ground, the contours may be drawn at 5-foot intervals, whereas for flat ground, 1-foot intervals may be better; in any case, contours should not be drawn so closely as to complicate the map unduly or interfere with its use. A convenient way to proceed is to make a map of the area on tracing cloth, and from this map make blue-line or black-line prints. Contours may be drawn on one of these, and it may be used to study the general layout of the drainage system. On other sheets, the plans may be developed roughly, routes and figures being marked in pencil directly on the map; as the design of the sewer system proceeds, there will be changes and corrections, and perhaps several possible plans may be carried nearly to completion before they are abandoned.

It is always necessary to study the map until the summits and depressions, and the ridges and valleys, are thoroughly impressed on the mind. It is sometimes of assistance to draw lines in red along the ridges, as the different drainage areas can be separated readily in this way if there is more than one. Blue lines through the valleys are of additional help in making distinct the details of the topography. Profiles of the streets should also be made. It will be an aid in locating the sewers to show on these profiles the depth and position of each basement. The proposed grade lines are drawn in pencil on the profiles.

When the final decisions are made, the sewer lines may be drawn on the tracing, making the map complete, and the grade lines of the sewers may be inked in on the profiles. Then the necessary detail maps may be prepared to show the exact locations of the sewers, with all specials and appurtenances.

GRADE LINES

24. Establishment of Grades.—The systems of main and lateral sewers should be so planned as to conduct the sewage to the outfall by the best available route, due attention being given to the questions of grade and location. The most direct route is generally—though not always—the best route, and is the one that will give the steepest grade. In some cases, where the grade is very steep, and where, by adopting a longer route, the sewer may still have a sufficient grade and may be more nearly parallel to the ground surface at a uniform depth, the longer route may be the better location. The available grades are governed largely by controlling points and by the general slope of the surface, to which they must, to some extent and in a general way, conform. The sewers should be laid with fairly uniform grades, however, and should not follow too closely the variations of the surface.

It must be remembered also that the necessary grade to create a velocity high enough to make the sewer self-cleansing and to prevent deposits, always must be maintained. Where the surface of the city or district is generally level and, consequently, the grades are very flat, the main sewers must often be laid by the shortest and most direct routes in order to obtain a sufficient grade or inclination, even though the depth of excavation at some points is greater than might be necessary over other routes.

25. Controlling Points.—There are several controlling points in fixing grades and depths. Thus, the upper portions of lateral sewers should, if possible, be at depths sufficient to accommodate the houses along the lines; at a certain point, there may be a deep basement that must be provided for; at another point, a branch sewer may have to be brought into the system at a given elevation; at still another point, the main sewer may have to be kept above a certain line, if possible, on account of quicksand and water, or other difficulties in construction. Above all, the elevation of the outlet must be such that the sewage will discharge freely at all times. In some

cases this will be a determining elevation. It is well to mark controlling points on the profiles in pencil, and then to harmonize as much as possible all the requirements that may be conflicting; it is always to be remembered that gradients must not be below the amount necessary to make the sewers self-cleansing.

TABLE I

MINIMUM GRADES FOR PIPE SEWERS

(*Velocity=2 ft. per sec.*)

Diameter of Sewer Inches	Grade Feet per 1,000 Feet
6	6.4
8	4.1
10	2.9
12	2.2
15	1.6
18	1.2
20	1.0
21	.95
24	.78
27	.67
30	.58
36	.46

26. Minimum Grades. The grades given in Table I are for a velocity of 2 feet per second when the sewer is half full, and may be taken as the reasonable minimum grades that will keep the sewers free from sediment and deposits. Where obtainable, however, velocities between 3 and 5 feet per second are desirable. A steeper grade is better at the upper end of a sewer because of the small volume of flow; occasionally a flatter grade must be used, but in such a case the possibility of stoppage and the cost of cleaning must be weighed against the advantages and cost of the flatter grade.

27. Depth of Sewer.—The depth at which a sewer should be located below the surface of the street will be determined principally by local conditions. Economical considerations would place the sewers as near the surface of the street as practicable, and storm sewers are usually constructed just beneath the surface. However, other considerations sometimes make it desirable that sewers should be located at considerable depths. Care should always be taken to place the sewer below the frost line. In order to avoid the water and

gas mains and the service pipes leading from them, which are necessarily placed below the frost line, it is the custom to locate the storm sewers just above these lines and the house sewers below them. The minimum depth of a house sewer, however, is generally determined by the positions of the house connections, some of which may be located in the basements. In residence districts, a depth of 8 to 10 feet below the surface is generally sufficient. In southern cities of small size, where there are no cellars under the houses, and where frost does not offer a problem, sewers are sometimes built only 3 or 4 feet deep. In general, from 12 to 15 feet is considered to be the maximum depth, except in special places, because the cost of construction increases rapidly with the depth, and the cost of house connections also becomes excessive.

28. Grade Lines at Junctions.—Sewers seldom, if ever, flow full, because the sizes are selected to provide for extreme conditions, which occur but rarely. Under usual circumstances, there is a large amount of air in the sewers, which tends to rise, the air currents thus flowing in the opposite direction to the sewage; and it is important in maintaining proper ventilation of the sewer not to obstruct the passage of the air. This feature requires special attention when one sewer joins another of larger size, or where the size of a sewer is increased. In order to fulfil the requirement best in such a case, it is desirable to have the upper parts, or *crowns,* of the two sections at the same level and, consequently, to place the lower part, or *invert,* of the larger pipe below the invert of the smaller sewer. The drop in the invert causes a loss in the available fall not less than the difference between the diameters of the two sections, and allowance should be made for the loss in determining the grades. Where the available fall is small, it may be necessary to put the inverts of the two sections at the same level in order to have no loss of head. Such an arrangement may at times interfere with the ventilation, but cannot always be avoided.

If the velocity of flow in a branch or a main sewer is reduced very much at a junction, solids may be deposited, especially

when the depth of the sewage is small. Hence, the grades of the inverts of the branch and main should be so related that for usual flows the velocities in the two sewers will be nearly equal and the elevations of the surfaces will also be about the same. In case the grades are adjusted for a small flow, the sewage may back into the branch for a great distance when the flow in the main is heavy. To prevent this, the grade of the branch is sometimes increased somewhat for a short distance from the junction.

REQUIRED CAPACITY OF SEWERS

QUANTITY OF SEWAGE IN SEPARATE SYSTEM

29. General Considerations.—The design of the sewers of a separate system is based on the quantity of sewage that is delivered to each sewer by the district tributary to it. This sewage will consist of that due to the use of the public water supply, that due to the use of private sources of water, and that due to leakage or infiltration of ground waters into the sewers. The quantity of sewage due to the use of the public water supply can be estimated quite closely; that from private supplies can also be estimated closely, but, as a rule, is not great enough to be of much importance; the amount of subsoil water that seeps into the sewers is dependent largely on local conditions and in some cases may be so large as to become the principal factor in the flow of sewage.

30. Ground Water and Subsoil Water.—An endeavor should be made so to construct sewers that they shall be watertight and allow no infiltration of ground water. It is difficult, if not impossible, to do this, particularly where quicksand or water-bearing soils are found; experience shows that, even with careful oversight and reasonably good work, sewers laid in such soils often gather a considerable amount of ground water. Cases have occurred where the entire capacity of the sewers was taken up by ground water that percolated through the joints, no capacity being left for carrying the sewage. It

is, therefore, wise to make some allowance for ground water in determining the required capacity of a sewer.

No definite rule can be given for determining the amount of subsoil water that may find its way into the sewers. Under favorable conditions, it may be as little as 5,000 to 10,000 gallons per mile of sewer per day; under less favorable conditions, even where the construction is fairly good, it may be five times this amount. Previous experience in similar places and under similar conditions is the only guide.

It is sometimes desirable to lay tile drains in the trenches with the sewers for the purpose of permanently draining the subsoil. When this is done, the water from these drains is usually discharged at a separate outlet, if possible, or into a storm sewer. If no outlet or storm sewer is available, the drains may be used during construction only, and stopped up when the sewer is completed.

31. Sewage Discharge and Water Supply.—Nearly all the water consumed by a city flows off as sewage. The exceptions include water used for sprinkling lawns, flushing streets (which usually flows into the storm sewers), fire fighting, some industrial uses, and a few other minor items. On the other hand, some industries may discharge large quantities of waste, which must be provided for. It should be noted, therefore, that the amount of water used by a city will be only a rough approximation of the amount of sewage that may be expected. Records of water supply are much more abundant then records of sewage flow, and, for this reason, estimates of sewage flow have been based quite largely on them. To be exact, however, the process of computing the amount of sewage that must be removed by sewers, allowances being made for estimated future population and for other unknown factors, consists largely of judging intelligently the relative importance of a number of factors.

32. Rate of Water Consumption.—Strictly speaking, there is no average rate of water consumption. The population of a city generally uses somewhere in the neighborhood of 100 gallons per capita per day, but in some cities, where meters

are in almost universal use, the water consumption does not exceed 70 to 75 gallons per person per day. European cities use less, perhaps from one-half to two-thirds of the amount used by American cities. In general, the amount of water used per person appears to be increasing. Probably the improved standards of living and the use of plumbing fixtures in greater number, and of a character that require more water, are largely responsible.

33. Variations in Sewage Flow.—The average daily water consumption is not of much value in sewer design because the consumption of water, and, consequently, the discharge of sewage vary greatly. In the selection of the size of a sewer, two kinds of fluctuations must be considered; namely, the daily fluctuations during the year, which determine what is called the *day maximum,* and the hourly fluctuations during the day, which determine what is known as the *hour maximum.* The hourly fluctuations in the water consumption and sewage discharge generally occur at rather regular periods during the 24 hours. The maximum hourly rate is usually reached an hour or two before noon; the rate may then be from one and one-half to two times the mean hourly amount, and in extreme cases much more.

There are generally two periods of excessive water consumption during the year, either of which may produce the maximum daily rate. One period occurs during the coldest weather, and the other during the warm, dry period that generally comes in late summer. The maximum rate in the cold weather period is the one that principally affects the sewage discharge. In severe weather, faucets are left running to prevent freezing, and there is, therefore, considerable waste. These and other conditions present during cold weather will, in most cases, produce the maximum rate of sewage discharge during the year, as nearly the entire volume of cold-weather water consumption will enter the sewer. During the warm and dry weather, much water will be used for sprinkling and similar purposes; this water, being absorbed by the ground and atmosphere, will not reach the sewer. The discharge for the maxi-

mum day may be 50 per cent., and in extreme cases more than 75 per cent., higher than the mean daily rate for the year.

34. Quantity of Sewage Determined by Measurement. Probably no record of sewage flow, even a continuous one, would really be serviceable in proportioning the sizes of the sewers of a separate system. Such records have been made, but the local conditions, the amount of ground-water infiltration, which cannot well be determined, and other factors give the records a limited usefulness as a basis of sewer design.

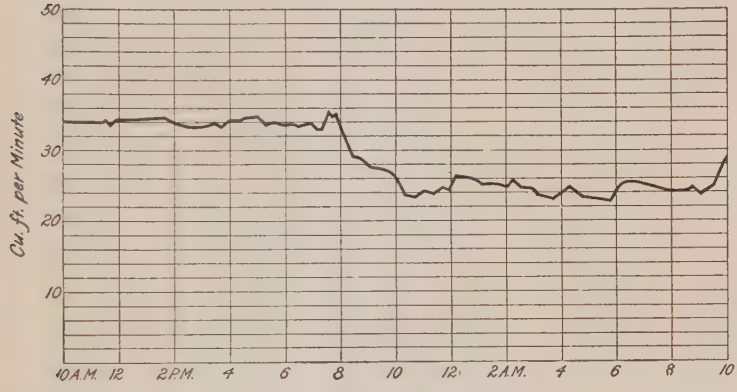

FIG. 6

In Fig. 6, taken from *Engineering News,* is a graphic record of sewer gaugings that were taken at 10-minute intervals over a period of 24 hours. These observations were made in a northern American city on a fairly cold day in February. The soil in which the sewers lie is a dense clay, and probably no subsoil water was mixed with the sewage. The diagram represents fairly the variations in sewage discharge for that time and place, and may be taken as a typical cold-weather diagram, where water is allowed to run at night to prevent freezing.

The following is a summary of the results:

Maximum rate.....35.28 cubic feet per minute
Average rate.....28.78 cubic feet per minute
Minimum rate.....22.15 cubic feet per minute

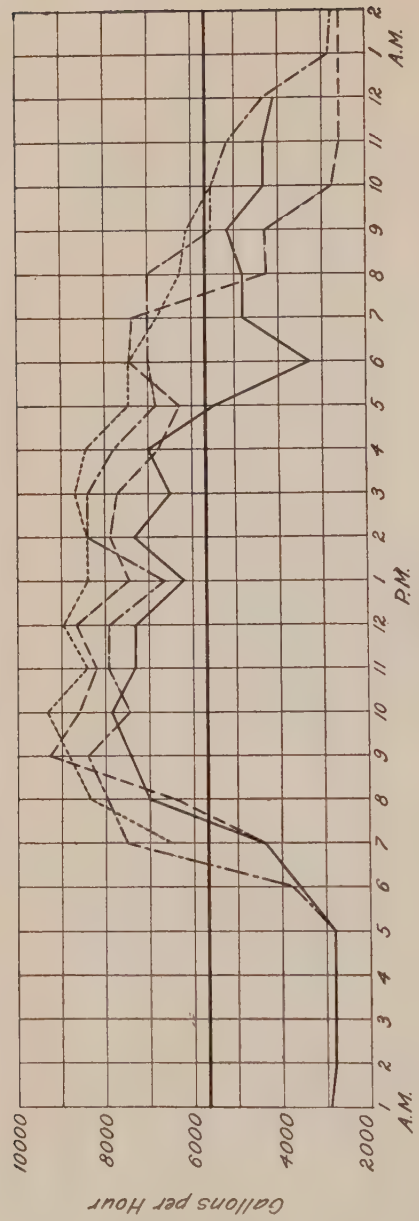
Gallons per Hour
10000
8000
6000
4000
2000
A.M.
1 2 3 4 5 6 7 8 9 10 11 12
P.M.
1 2 3 4 5 6 7 8 9 10 11 12
A.M.
1 2

FIG. 7

If the average rate is taken as 100, the relative rates are as follows:

Maximum rate	123
Average rate	100
Minimum rate	77

In Fig. 7, from Ogden's *Sewer Design,* are shown records for four consecutive summer days, made at Chautauqua, New York. The diagram is fairly typical of ordinary conditions. The heavy horizontal line shows the average flow.

The following is a summary of the results:

Maximum rate	20.5 cubic feet per minute
Average rate	12.7 cubic feet per minute
Minimum rate	5.8 cubic feet per minute

If the average rate is assumed to be 100, the relative rates are:

Maximum rate	162
Average rate	100
Minimum rate	46

35. Assumptions in Design.—A common practice among American engineers is to proportion the sewers of a separate system so that, when running half full, they will discharge a quantity of sewage equal to the maximum hourly water consumption, this maximum being taken at 175 per cent. of the average, and allowance being made for future increase in population. The remaining capacity is reserved for extreme variations in flow, and for purposes of ventilation. The average daily flow may be defined as that flow reasonably to be expected when the territory is fairly well developed, and all buildings are connected with the sewer. Another common basis of design is to construct the sewers so that they will flow half full with twice the average future flow 25 years hence, plus allowance for ground-water infiltration.

QUANTITY OF STORM WATER

RATE OF RAINFALL

36. General Remarks.—During a storm, the intensity of rainfall seldom, if ever, remains uniform for a long time, but the precipitation generally varies from a mere drizzle to a heavy downpour. Evidently, the average rate of rainfall, which is found by dividing the total precipitation by the duration of the entire storm, will be less than the maximum rate for a short period. For example, when 6 inches of rain fall in 12 hours, 3 inches of it may be precipitated in 2 hours, and, perhaps, 2 inches in 30 minutes. Thus, the average rate during the entire storm is $\frac{6}{12}$, or $\frac{1}{2}$, inch per hour, while the rate during the 30 minutes of heaviest rainfall is $2 \div \frac{1}{2}$, or 4, inches per hour. The greatest intensity of rainfall persists for only a short period; and the shorter the period, the more intense the rainfall is likely to be.

In designing a storm-water sewer it is necessary to consider the conditions which tend to produce a maximum flow through the sewer. Generally, the capacity of a sewer at a given point is taxed most severely by a heavy downpour, after the ground has become soaked, that lasts just long enough for the water from the most remote parts of the district to reach the sewer and flow through it to the point under consideration. A storm lasting longer than this period has a lower intensity and, therefore, the volume of water flowing through the sewer at one time will not be so large. If, on the other hand, the storm has a shorter duration, the water from the nearby points will have left the sewer before the water from the more distant points reaches the sewer. A storm moving toward the sewer will give the greatest volume of flow, because the water from the remote parts of the district will reach the sewer relatively sooner and the effect will be the same as though the storm had a shorter duration.

The chief condition to be considered in designing a storm sewer is, therefore, the maximum intensity of the precipitation

for a period of time sufficient for the water from the most remote parts of the district to reach the sewer and flow to the point under consideration.

37. Records of Rainfall.—Records of daily, monthly, and yearly rainfall are numerous; but useful as such records are for some purposes, they have little value in the design of sewers. A locality having a large annual rainfall may be subjected only to long-continued drizzling rains and so may be served by small sewers; while another district, which has a much smaller annual rainfall, may require larger sewers because of heavy rains. The records of storms, as generally reported, give the total precipitation for the entire storm, and possibly the duration of the storm, but they do not usually give the maximum rate of rainfall for short periods like 5 or 10 minutes. To obtain records of rainfall intensity, which are needed in the design of sewers, automatic recording rain gauges, that note the time as well as the amount of precipitation, are required. Such gauges are generally located in the larger cities and also in the more important stations of the United States Weather Bureau, and their records are available.

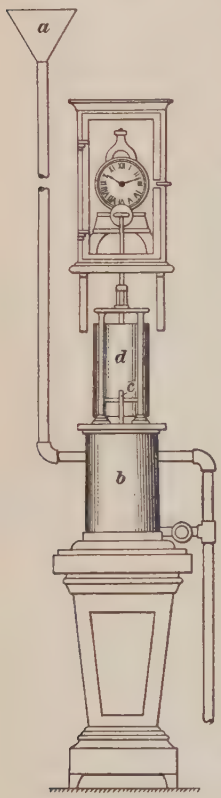

FIG. 8

38. Self-Registering Rain Gauge. One of the most satisfactory recording gauges is that devised by Fitzgerald, an outline cut of which is shown in Fig. 8. The rain is collected in a funnel *a*, 14.85 inches in diameter, and passes into a receiving chamber *b*, which contains a float. As the rain water enters the chamber, the float rises; and, by means of a pencil *c*, attached to the float, a record of the rainfall is marked on a chart wound on the cylinder *d*, the cylinder being rotated at a uniform rate by clockwork.

FORMULAS FOR MAXIMUM RATE OF RAINFALL

39. General Formula.—Inasmuch as the rate or intensity of rainfall varies inversely with the duration of the storm, it is possible to construct a formula, based on past rainfall records that will express approximately the maximum rate of rainfall that may be expected during a storm lasting for a given interval of time. Most formulas giving the relation between the duration of a storm and the maximum intensity of rainfall are written in the form

$$i=\frac{a}{t+b}$$

in which i = rate of rainfall, in inches per hour;
a and b = constants;
t = duration of storm, in hours.

40. Talbot's Formulas.—The formula proposed by Professor Talbot for the maximum rate of ordinary rainfall in the part of the United States east of the Rocky Mountains is

$$i=\frac{1.75}{t+.25} \qquad (1)$$

This formula is derived from the general formula given in the preceding article by substituting 1.75 for a and .25 for b. When the duration of the storm is short, it is often more convenient to express the time in minutes instead of hours. Then the formula is

$$i=\frac{105}{t_1+15} \qquad (2)$$

in which i = intensity of rainfall, in inches per hour;
and t_1 = duration of storm, in minutes.

Formula **2** is obtained from formula **1** simply by multiplying each constant by 60 and replacing the time in hours by the time in minutes, as these changes are equivalent to multiplying both the numerator and denominator of the fraction by 60.

For the maximum rate of rare rainfall, Talbot's formulas are

$$i=\frac{6}{t+.5} \qquad (3)$$

and

$$i=\frac{360}{t_1+30} \qquad (4)$$

in which i, t, and t_1 have the same meanings as before. The curves in Fig. 9 are plotted from Talbot's formulas.

The values given by formula **1** or **2** apply reasonably well throughout most of the northern states east of the Rocky Mountains. It must be kept in mind, however, that the frequency with which the estimated rate is likely to be reached or exceeded by actual storms will vary considerably in different sections of the country; for instance, violent storms occur much more

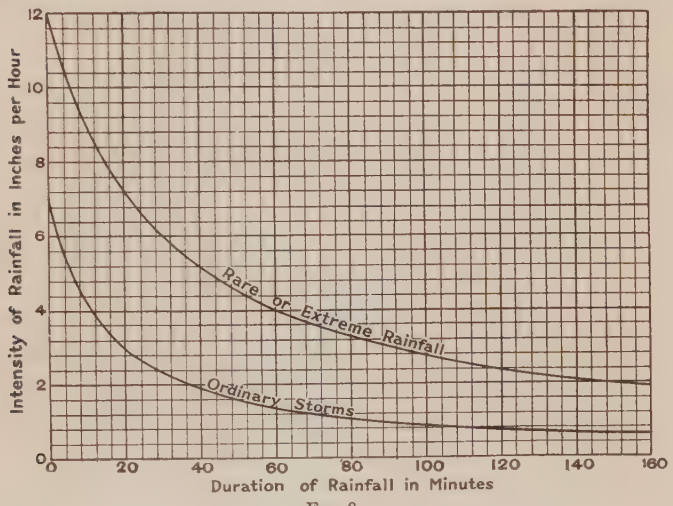

FIG. 9

frequently in the southern Atlantic states than in the central or western states. According to Professor Talbot, it is probable that rainfall intensities reaching the value given by his formula for maximum rate of ordinary rainfall will not occur at a given point more than two or three times in 10 years, and that the value found by his formula for rare rainfall will not be exceeded oftener than once in about 50 years. Since it is generally more economical to let a sewer be overtaxed once in 10 years or so than to have it entirely too large for ordinary conditions, formulas **3** and **4** have little practical value in the design of sewers.

EXAMPLE.—What is the maximum rate of ordinary rainfall, according to Talbot's formula, for a storm lasting 30 minutes?

SOLUTION.—Since the duration is given in minutes, formula **2** is used. Here, $t_1=30$, and $i=\frac{105}{t_1+15}=\frac{105}{30+15}=2.33$ in. per hr. Ans.

41. Other Formulas for Rainfall.—As a basis for the design of sewers in some localities, the results given by Talbot's formulas for ordinary rainfall are too low. In places where the rainfall is frequent and heavy, as, for example, in the southern Atlantic states, the following formulas for the maximum rate of occasional rainfall appear to be satisfactory.

$$i=\frac{2.25}{t+.3} \qquad (\mathbf{1})$$

$$i=\frac{135}{t_1+18} \qquad (\mathbf{2})$$

in which i, t, and t_1 have the same meanings as in Art. **40.** Rates of rainfall given by either of these formulas should not be exceeded at any point in the United States oftener than once in about 5 years. Unless otherwise stated, these formulas will be used in the solutions of problems in this Section, although they are really no more general than Talbot's formulas.

In Table II are given the values of i computed by formula **1** or **2** for various values of t and t_1 ranging from 5 minutes to 3 hours. For intermediate values of t or t_1 the corresponding value of i can be found by interpolation.

EXAMPLE.—Determine the maximum rate of occasional rainfall for a storm whose duration is (*a*) 30 minutes; (*b*) 2 hours 40 minutes.

SOLUTION.—(*a*) The duration of the storm is .5 hr. or 30 min., and the corresponding intensity of rainfall in Table II is 2.81 in. per hr. Ans.

(*b*) In this case, the duration of the storm is 160 min., which is not found in the second column of Table II. The values on each side of 160 are 150 and 165, and the corresponding rainfall rates are .80 and .74. Hence, the required rate is $.80-\frac{160-150}{165-150}(.80-.74)=.76$ in. per hr. Ans.

42. For periods varying between 10 minutes and 2 hours, formulas of the form $i=\frac{a}{t+b}$ are fairly accurate, but for shorter or longer storms, the results are generally too low. In localities east of the Mississippi River, the following formula,

from *American Sewerage Practice* by Metcalf and Eddy, may be used as a reasonable basis of design.

$$i=\frac{a}{\sqrt{t_1}}$$

in which i = rate of rainfall, in inches per hour;
a = constant whose value is between 10 and 15, depending on the locality;
t_1 = duration of storm, in minutes.

TABLE II

MAXIMUM RATES OF OCCASIONAL RAINFALL

Duration of Storm		Rate of Rainfall Inches per Hour	Duration of Storm		Rate of Rainfall Inches per Hour
Hours	Minutes		Hours	Minutes	
	5	5.87	1.00	60	1.73
	8	5.19		65	1.63
	10	4.82		70	1.53
	12	4.50	1.25	75	1.45
.25	15	4.09		80	1.38
	18	3.75		85	1.31
	20	3.55	1.50	90	1.25
	22	3.38		100	1.14
	25	3.14		110	1.05
	28	2.93	2.00	120	.98
.50	30	2.81		130	.91
	35	2.55		140	.85
	40	2.33	2.50	150	.80
.75	45	2.14	2.75	165	.74
	50	1.99	3.00	180	.68
	55	1.85			

For conditions in the northeast sections of the country, values obtained by taking a as 12 are not likely to be exceeded oftener than once in about 10 years.

EXAMPLE.—Solve the example of Art. **41** by the use of Metcalf and Eddy's formula, in which a is taken as 12.

SOLUTION.—(a) Here, t_1=30 and

$$i=\frac{a}{\sqrt{t_1}}=\frac{12}{\sqrt{30}}=2.19 \text{ in. per hr. Ans.}$$

This result is less than the value found by means of Table II.

(b) In this case, t_1=160 and

$$i=\frac{12}{\sqrt{160}}=.95 \text{ in. per hr. Ans.}$$

This result is somewhat greater than the corresponding value in Table II.

EXAMPLES FOR PRACTICE

1. Find from Table II the maximum rate of occasional rainfall for a storm lasting (a) 40 minutes; (b) 1 hour 45 minutes.

Ans. { (a) 2.33 in. per hr.
(b) 1.1 in. per hr.

2. Determine, by means of Fig. 9, the maximum rate of rare rainfall for a storm whose duration is 30 minutes. Ans. 6 in. per hr.

3. Compute, by the formula $i=\frac{12}{\sqrt{t_1}}$, the maximum rate of occasional rainfall for a storm lasting 4 hours 25 minutes. Ans. .74 in. per hr.

CONSIDERATIONS AFFECTING SEWER DESIGN

43. Allowance for Violent Storms.—The very heaviest storms are extremely rare, although unusually hard downpours occur at intervals of from 5 to 10 years. It is not customary, nor is it generally desirable, to design a sewerage system with a capacity sufficient to remove promptly, by means of the sewers alone, all the water from an extraordinary storm. Most sewers are designed to care for future improvements of the ground surface, which may not take place for 10 or more years, and during this period the capacity will likely be enough even for very rare storms. After the surface has been improved, a little storm water can be conveyed in the street gutters, and the sewers relieved to some extent in this way. Furthermore, sewers with capacities sufficient to meet the

requirements of excessive storms may not be advantageous as conduits for usual conditions. Besides it may even be better to allow an occasional overflow, perhaps once in ten or more years, than to expend too much money on the construction of a sewer that will be taxed to capacity only once or twice in its lifetime.

44. Maximum Rainfall Rate.—When rainfall records obtained by a recording gauge are available for the locality, they should be used as a basis in determining the maximum rate of precipitation for storms of various durations. From these records, curves can be plotted, or formulas derived and tables prepared, to represent the maximum intensities of rainfall for ordinary storms, for unusual storms, and for very rare storms. The best formula for rainfall may usually be obtained by substituting suitable values for a and b in the formula of Art. **39.** The values of the maximum rates of rainfall for storms of various durations may then be tabulated as in Table II, which is derived by applying formula **1** or **2,** Art. **41.**

In Fig. 10 are shown rainfall-intensity curves that were proposed as a basis for sewer computations in the city of New York. These curves, which are derived from gaugings and records extending over a period of about 50 years, give the values for durations of storms from 5 to 120 minutes; each curve indicates the maximum rate that may be expected to occur but once in the period. For example, let it be required to determine the rate of precipitation to be used in designing a sewer for a storm that may occur once every 25 years, when 20 minutes are required for the storm water from the remote sections of the district to reach the sewer. Since the 25-year curve intersects the vertical line representing a duration of 20 minutes on the horizontal line indicating 4.4, the required intensity of rainfall is 4.4 inches per hour. In Fig. 11 are shown curves representing the maximum rates of rainfall for ordinary storms, for extraordinary storms, and for extremely rare storms, derived from Philadelphia rainfall records covering a period of 25 years. When records for the locality are

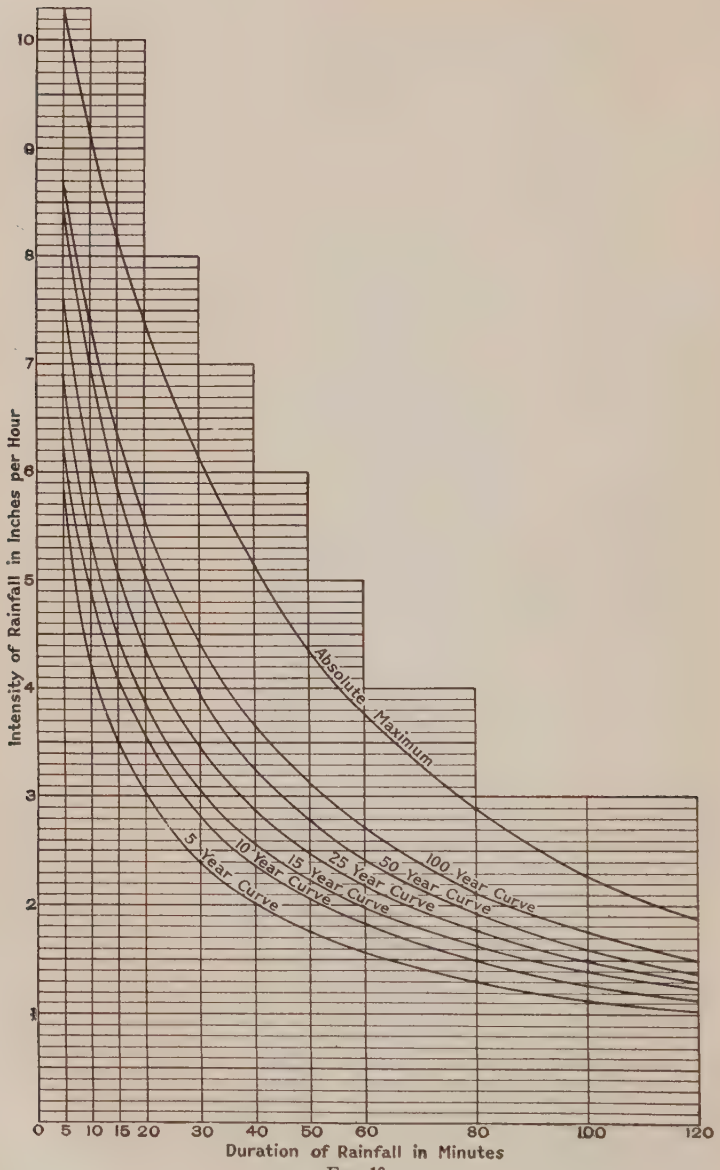
Intensity of Rainfall in Inches per Hour
10
9
8
7
6
5
4
3
2
1
0 5 10 15 20 30 40 50 60 80 100 120
Duration of Rainfall in Minutes
Absolute Maximum
100 Year Curve
50 Year Curve
25 Year Curve
15 Year Curve
10 Year Curve
5 Year Curve

FIG. 10

not available, general formulas for similar conditions may be used.

45. Rainfall in Cubic Feet per Second per Acre.—It will be well to mention here that the rate of rainfall in inches per hour corresponds very closely with the number of cubic feet of rain falling on one acre in a second. This is shown as fol-

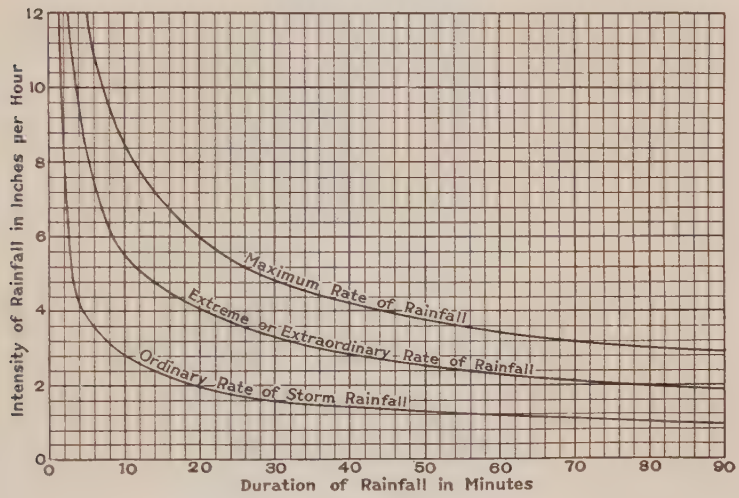

FIG. 11

lows: Since there are 43,560 square feet in an acre, the volume of a prism, 1 inch, or $\frac{1}{12}$ foot, high and 1 acre in cross-sectional area, equals $\frac{1}{12} \times 43{,}560 = 3{,}630$ cubic feet. Hence, a rainfall of 1 inch per hour gives 3,630 cubic feet of water per acre per hour, which equals $\frac{3{,}630}{3{,}600} = 1.0083$ cubic feet per acre per second. Therefore, in all computations relating to the required capacities of sewers, the number of cubic feet of rain water falling on an acre in each second may, without material error, be taken equal to the intensity of rainfall, in inches per hour.

46. Rainfall Rate for Sewer Design.—The maximum intensity of rainfall depends on the duration of the storm. As already explained, the maximum flow in a sewer generally

occurs when the duration of the storm is the same as the time required for the water from the remote parts of the district to reach the sewer. Therefore, the rate of rainfall to be used in determining the required capacity of the sewer depends primarily on the proper choice of that time. After the duration of the storm has been selected, the corresponding intensity of rainfall is chosen by a study of rainfall records of the past, or by applying a general formula.

47. Time of Concentration.—The interval of time required for the water from the most remote parts of a district to reach a given point in the sewer is called the *time of concentration*. It is made up of two periods, namely, the time required for the water to reach the sewer inlet, and the time required for it to flow down the sewer to the point under consideration. The time of flow in the sewer depends only on the size, slope, and smoothness of the sewer, and can easily be computed by applying the principles of hydraulics. The time required for the storm water to reach the inlet cannot be determined accurately because it depends largely on the topography of the district, being affected not only by the size and shape of the district but also by the slope and character of the ground surface. Metcalf and Eddy state that in cities it is seldom less than 3 minutes or more than 20 minutes.

The velocity of the storm water is less over nearly level ground than over ground having a greater slope. Also, the flow will be more rapid over the surface of a paved district than over the surface of an unpaved district, and more rapid over a smooth lawn than over a wooded tract. If, to reach the sewer, the water must flow through rough and crooked channels, the time required will be much more than if the water is led by direct courses through smooth channels. It must be borne in mind, however, that the determination of the time of concentration should not be based on present conditions of the ground surface, but on development that may reasonably be expected before the sewer must be replaced by a larger one.

RUN-OFF

AMOUNT OF RUN-OFF BY RATIONAL METHOD

48. Factors Affecting Run-Off.—Generally the problem of estimating storm-water run-off is an indeterminate one because the results are affected by so many factors that depend on judgment. Besides the time of concentration and the maximum intensity of rainfall, other factors that have an important influence on the amount of water to be handled by a storm water sewer are the development of the district as regards streets, pavements, buildings, etc., and the condition of the ground surface. The problem is made even more difficult by the fact that the design must be based on conditions that may reasonably be expected at some future time.

49. Rational Formula.—The rational method of estimating run-off is now used quite extensively in the design of storm water and combined sewers for the larger cities. This method is based on the relation

$$Q = C\,i\,A$$

in which Q = total run-off from district, in cubic feet per second;
C = coefficient;
i = intensity of rainfall, in inches per hour or cubic feet per second per acre;
A = area of district, in acres.

The intensity of rainfall i, corresponding to a duration of storm equal to the time of concentration, is found by means of a formula, chart, or table; and the area A of the district can be determined readily by measurement. The run-off Q is the amount of storm water that enters the sewer during a period equal to the duration of the storm, and is called the *contemporary flow*. The coefficient C represents, for the given conditions, the proportion of the rainfall that reaches the sewer in a period of time equal to the time of concentration, that is, the ratio of the contemporary flow to the rainfall, and is called

the *coefficient of storm flow.* It is so affected by local conditions that its value cannot be given accurately for districts of different character. It must be determined for each particular case by a careful study of various factors.

50. Factors Affecting Contemporary Flow.—The proportion of the rainfall that reaches the sewer during the time of concentration varies according to the area, slope, and condition of the surface, and also according to the nature of the subsoil. If the district is large and its surface comparatively level and rough, considerable time will be required for the

TABLE III

RELATION OF FULLY IMPERVIOUS SURFACE TO TOTAL AREA ACCORDING TO DENSITY OF POPULATION

Class of District	Average Number of Persons per Acre	Percentage of Fully Impervious Surface			
		Roofs	Improved Streets	Unimproved Streets, Yards, Etc	Total
c	25	14.0	7.0	4.3	25.3
d	32	18.0	10.2	5.0	33.2
e	40	22.5	14.7	5.4	42.6
f	50	28.0	19.0	5.6	52.6

storm water to flow to the sewer, and there may be a large amount of evaporation and absorption. Moreover, in a large district, the maximum rate of precipitation for the storm may not extend throughout the entire watershed. Hence, the contemporary flow per acre is much less for a large, flat district than for a small, hilly one.

Wooded tracts, cultivated lands, or those covered by luxuriant vegetation retain a greater part of the rainfall, and yield more slowly what they do not retain than smoothly cut lawns or areas, such as sidewalks or streets, that are devoid of vegetable growths. The latter classes of surfaces will, therefore, give a greater flow. In districts that are closely built up, the greater part, and often the whole, of the surface on which

the rain falls consists of paved streets, courts, and the roofs of buildings, all of which offer nearly impervious surfaces to water; in addition, the systems of surface drains, troughs, and gutters quickly convey the water to the sewers. As a result, a large part of the rainfall is promptly delivered to the sewers. Frozen ground may give a surface flow practically equal to the rainfall; and if the rain occurs simultaneously with the melting of the snow, the surface flow may considerably exceed the rainfall.

The amount of contemporary flow is also affected by the nature of the soil. Loose, porous soils, as sand or loam, readily

TABLE IV

RATIO OF TOTAL RUN-OFF TO RAINFALL FOR VARIOUS SURFACES

Surface	Ratio
Roofs	.70 to .95
Concrete or asphalt pavements	.85 to .90
Stone, brick, or wood-block pavements, in good condition	.75 to .85
Stone, brick, or wood-block pavements, in poor condition	.40 to .70
Macadamized roads	.25 to .60
Gravel roads and walks	.15 to .30
Unpaved streets, vacant lots, etc.	.10 to .30
Parks, lawns, gardens, meadows, etc.	.05 to .25
Wooded and fertilized lands	.01 to .20

soak up a large proportion of the rainfall, whereas hard-packed and impervious soils, like clay and cemented material, take up much less of the rainfall and, consequently, give a much greater surface flow.

51. Determination of Coefficient of Storm Flow.—Gaugings in sewers during the flow of storm water indicate that, after the ground has become saturated, the coefficient of storm flow is practically constant for a given district for all rates of precipitation; and, in the case of roofs and well-paved areas,

the effect of saturation is slight. However, as already stated, the value of the coefficient depends largely on the character of the surface and subsoil, and gaugings sufficient to establish accurate values for different conditions have never been made.

TABLE V

COEFFICIENT OF STORM FLOW FOR DRAINAGE DISTRICTS

Class of District	Population per Acre	Character of District	Coefficient		
			Minimum	Maximum	Mean
a	10	Unimproved suburbs .	.06	.12	.09
b	20	Improved suburbs, unpaved	.10	.18	.14
c	25	Macadamized residence suburbs	.15	.20	.18
d	32	Ordinary suburban districts, roughly paved	.20	.27	.24
e	40	Built-up, paved districts	.26	.34	.30
f	50	Closely built-up and well-paved districts .	.32	.42	.36
g		Densely built-up and exceedingly well-paved districts	.42	.56	.50
h		Solid stretches of roofs, walks, and pavements, in perfect condition .	.56	.75	.65

Such reliable information as is available is briefly discussed in the following articles.

52. Kuichling's Values.—When there is no suitable information from which the coefficient of storm flow can be estimated, an approximate result can be based on the percentage of the area of the district that may be considered impervious to water. In Table III, which was prepared by Kuich-

ling, are given the percentages of impervious surfaces of various classes for different densities of population. The values in the last column represent the total proportion of the rainfall that will reach the sewer. These percentages are not, however, the measure of the contemporary flow, as some of the water may come to the sewer after the storm has abated. In order to compare the values in this and following tables, each class of district is given a distinguishing letter.

TABLE VI

TALBOT'S RATIOS OF TOTAL RUN-OFF TO RAINFALL

Class of District	Character of District	Ratio
b	Suburban districts, sewered, but not paved	.20
d	Suburban districts, paved and sewered	.30 to .40
f	Closely built-up districts, paved and sewered	.40 to .50
h	Roofs of buildings	Nearly 1

Kuichling has also estimated the ratio of the total run-off to the rainfall for various kinds of surfaces. His values are given in Table IV.

Gaugings in London, England, indicate that the amount of water reaching the sewer during the time of concentration is from 60 to 80 per cent. of the entire run-off. From Kuichling's values for the percentages of impervious surfaces and these percentages of run-off, the values of the coefficient of storm flow given in Table V are derived. For example, in a district of class *d*, the minimum value of the coefficient equals $.6 \times 33.2 = 20$ per cent., or .20, and the maximum value equals $.8 \times 33.2 = 27$ per cent., or .27. For districts of classes *g* and *h*, the percentages of impervious surfaces are taken as 70 and 94, respectively; in districts of classes *a* and *b*, the conditions vary so greatly that different percentages of impervious

surfaces are taken in computing the minimum and maximum values of the coefficient.

53. Talbot's Values.—The ratios of total run-off to rainfall for city districts, as derived by Professor Talbot, are given in Table VI. His values for the proportion of rainfall that is contemporary flow are given in Table VII. It will be seen

TABLE VII

TALBOT'S RATIOS FOR CONTEMPORARY FLOW

Class of District	Population per Acre	Ratio
a	10	.10
b	20	.20
c	40	.30
f	50	.40
g	Denser	.50 or more

that there is a close agreement between the values in Tables V and VII for corresponding districts.

54. Other Values of Coefficient.—In some districts, the percentages of the area that have various degrees of impermeability can be determined accurately, and the resultant run-off ratio for the entire district can be computed by taking the sum of the products of these percentages and the corresponding run-off ratios for the kinds of surfaces. For example, suppose that 20 per cent. of the area consists of roofs, from which the ratio of run-off to rainfall is .90; 20 per cent. of the area is pavement having a run-off ratio of .85; 55 per cent. is occupied by unpaved streets, vacant lots, lawns, parks, gardens, etc., whose run-off ratio is .20; and 5 per cent. is wooded with a run-off ratio of .10. Then, the resultant run-off ratio for the entire district may be taken equal to $.20\times.90+.20\times.85+.55\times.20+.05\times.10$, or .465. The coefficient of storm flow is equal to the run-off ratio multiplied by a factor, varying from .6 to .8, which represents the proportion of the run-off that reaches the sewer during the time of concentration.

Sometimes, it is further assumed that the maximum intensity of rainfall occurs at the beginning of a storm before the ground surface has become saturated and that the coefficient of storm flow increases with the time of concentration. However, such an assumption is not safe when the time of concentration is relatively short, and it is usually better to suppose that the ground is saturated before the storm begins.

55. Selection of Coefficient.—There is no well-defined distinction between the districts of different classes listed in Table V; the classes merge into one another, and judgment and care must be exercised in selecting the class to which any district belongs. After the district has been classified, it is necessary to decide on the value of the coefficient of storm flow. The coefficients in the table represent, for the given conditions, the proportion of the rainfall that is assumed to flow away during a period equal to the time of concentration. The rate of flow based on that assumption probably will not be maintained throughout the entire period, but may be attained for a short time. Hence, it may be used as a guide in determining the required capacity of the sewer.

The size and slope of the district affect to some extent the value of the coefficient, because they influence greatly the time of concentration and the amount of evaporation and absorption. In general, the maximum value of C should be used for a small district having steep slopes, and the minimum value should be taken for large and comparatively level districts. A good designer should take every opportunity to compare actual rainfall rates and corresponding storm-flow measurements with which he is familiar, in order to acquire a conception of the coefficients of storm flow that will apply approximately to different kinds of districts. In this Section, the mean values given in Table V will be used for C, unless otherwise stated.

56. Illustrative Examples.—In applying the formula $Q = C\,i\,A$, which is given in Art. **49,** it is necessary to choose by judgment the proper values of C and i. In practice, they would be based on a study of past rainfall records for the locality and the topography and character of the district, but

in the following examples the necessary assumptions will be made to permit of their selection.

EXAMPLE 1.—What is the contemporary flow in cubic feet per second from a district of class *e*, which has an area of 4 acres, if the time of concentration is 15 minutes?

SOLUTION.—The maximum rate of rainfall, *i*, for a storm lasting 15 min. is found, from Table II, to be 4.09 in. per hr. From Table V, the mean value of *C*, the coefficient of storm flow, for a district in class *e*, is .30. Hence, by the formula of Art. **49**,

$$Q = C\,i\,A = .30 \times 4.09 \times 4 = 4.9 \text{ cu. ft. per sec. Ans.}$$

EXAMPLE 2.—For a given district, it is assumed that the coefficient of storm flow varies with the time of concentration in the following manner:

Time Minutes	Coefficient	Time Minutes	Coefficient
10	.45	30	.60
15	.50	45	.65
20	.55	60	.70

It is required to prepare a table giving the maximum flow per acre for storms of various durations.

SOLUTION.—In the following table, the durations of storms are given in the first column; the corresponding rates of rainfall, *i*, taken from

Duration of Storm Minutes	Rate of Rainfall Inches per Hour	Coefficient of Storm Flow	Maximum Flow Cubic Feet per Second per Acre
10	4.82	.45	2.17
15	4.09	.50	2.05
20	3.55	.55	1.95
30	2.81	.60	1.69
45	2.14	.65	1.39
60	1.73	.70	1.21

Table II, in the second column; the values of the coefficient *C* assumed for the district, in the third column; and the products of the rate of rainfall and the coefficient, in the last column. These products are

the required values of the maximum flow in cubic feet per second per acre. The flow for any intermediate duration of storm may be found by interpolation. A curve representing the relation between the duration of the storm and the flow may be constructed from the values in the table. Such a curve is not necessarily smooth; in this case, for instance, there will be a sharp break at the point where the duration is 20 minutes.

EXAMPLES FOR PRACTICE

1. What is the contemporary flow in cubic feet per second per acre from a district in class *d*, if the time of concentration is 18 minutes?

Ans. .9 cu. ft. per sec.

2. A district in class *g* has an area of 3.5 acres. Find the contemporary flow during a storm lasting 12 minutes.

Ans. 7.9 cu. ft. per sec.

3. The area in a district is composed of the following surfaces: 15 per cent. roofs with a run-off ratio of .92; 25 per cent. pavement with a ratio of .80; 5 per cent. gravel walks with a ratio of .20; 45 per cent. vacant lots, lawns, and gardens with a ratio of .15; and 10 per cent. fertilized land with a ratio of .10. Find the resultant run-off ratio for the entire district. Ans. .4255

FORMULAS FOR RUN-OFF

57. Use of Formulas.—In many places, the amount of storm water that is expected to reach the sewers is computed by empirical formulas instead of by the rational method. Some of these formulas are based on local conditions and are local in application; others are based on studies extending over many years and are applicable to many localities, provision being made for local conditions by means of coefficients, which vary with the rainfall, the size and slope of the district, etc. In many places in the southeastern part of the United States, a flat rate of 2 cubic feet per second per acre is assumed to be the probable maximum flow, and sewers are designed on that basis. In most cases, however, formulas are used in which weight is given to those local conditions that may influence the run-off. The two formulas most generally used are those of Bürkli-Ziegler and McMath.

58. Bürkli-Ziegler Formula.—Of the various formulas proposed for computing the required capacities of storm-water

sewers, one of the most reliable is that of Bürkli-Ziegler; it is

$$F = C_1 i \sqrt{\frac{S}{A}} \qquad (1)$$

in which F = maximum flow, in cubic feet per second per acre;

C_1 = coefficient for proportion of rainfall that will reach sewer during time of concentration;

i = maximum intensity of rainfall, in inches per hour;

S = average surface slope toward and along sewer in feet per thousand feet;

A = area of district, in acres.

The total flow from the district is found by multiplying F, which is the flow per acre, by the area of the district. Hence,

$$Q = F A = C_1 i \sqrt[4]{S A^3} \qquad (2)$$

in which Q = maximum flow from district, in cubic feet per second.

In Table VIII are given the values of the expression $\sqrt[4]{S A^3}$ for various values of S and A. If the values of S and A for the district in question are not in the table, the required value of $\sqrt[4]{S A^3}$ may be found by interpolation.

59. Values in Bürkli-Ziegler Formula.—The value of the coefficient C_1 in the Bürkli-Ziegler formula ranges from .50 in rural sections and suburbs to .90 in well built-up districts. The average value, which is about .75, will be used in this Section, unless otherwise stated.

In climates where the intensity of rainfall varies greatly with the duration of the storm, the length of a typical storm is decided on and the corresponding rate of rainfall is found from Table II or by some other method. The European practice is to give i a value ranging from 1.75 to 2.25 inches per hour, but in recent American practice, the value of i is taken between 2 and 3.5; even higher values are used when the sewers are designed to carry all the storm water. In St. Louis, Missouri, rates ranging from 3 to 3.5 were used. Observations

taken in Rochester, New York, during storms lasting less than one hour indicate that, for those conditions, the greatest run-off is obtained from storms lasting 51 minutes. For this duration of a storm, the maximum intensity in Table II is 1.96, or, say, 2, inches per hour. For American practice, the mean value of i is about 2.75.

TABLE VIII

VALUES OF $\sqrt[4]{S A^3}$

A	S=2.5	S=5	S=10	S=15	S=20	S=25	S=50	S=100
40	20.00	23.78	28.28	31.30	33.64	35.57	42.29	50.30
60	27.10	32.24	38.34	42.43	45.59	48.21	57.33	68.17
80	33.64	40.00	47.57	52.64	56.57	59.81	71.13	84.59
100	39.76	47.29	56.23	62.23	66.87	70.71	84.09	100.00
120	45.59	54.22	64.47	71.35	76.67	81.07	96.41	114.65
160	56.57	67.27	80.00	88.53	95.14	100.60	119.63	142.26
200	66.87	79.53	94.57	104.66	112.47	118.92	141.42	168.18
300	90.64	107.79	128.19	141.86	152.44	161.19	191.68	227.95
400	112.47	133.74	159.05	176.02	189.15	200.00	237.84	282.84
500	132.96	158.09	188.02	208.09	223.61	236.44	281.17	334.37
600	152.44	181.28	215.58	238.58	256.37	271.08	322.37	383.37
800	189.15	224.92	267.50	296.03	318.11	336.36	400.00	475.68
1,000	223.61	265.90	316.23	349.96	376.06	397.64	472.87	562.34
1,200	256.37	304.84	362.57	401.24	431.17	455.90	542.16	644.74
1,500	303.08	360.39	428.62	474.34	509.71	538.96	640.93	762.20
2,000	376.06	447.21	531.83	588.57	632.46	668.74	795.27	945.74
2,500	444.57	528.68	628.72	695.79	747.67	790.57	940.15	1,118.03

EXAMPLE 1.—A district, whose area is 160 acres, has an average slope of 2 feet in 100 feet. If C_1 is taken as .54 and i as 2.5 inches per hour, what will be the required capacity of the sewer, in cubic feet per second, by the Bürkli-Ziegler formula?

SOLUTION.—The slope is 2 ft. per 100 ft., or 20 ft. per 1,000 ft., and, therefore, S=20. From Table VIII, the value of $\sqrt[4]{S A^3}$ when S=20 and A=160 is 95.14. Hence, by formula **2**, Art. **58**,

$$Q=C_1 i \sqrt[4]{S A^3}=.54\times2.5\times95.14=128 \text{ cu. ft. per sec. Ans.}$$

EXAMPLE 2.—Find the flow from a district that is similar to that in example 1 but has an area of 125 acres.

SOLUTION.—In this case, the given area is not found in the column of the table headed A, and the value of $\sqrt[4]{S A^3}$ must be determined by interpolation. The required number equals

$$76.67+\frac{125-120}{160-120}(95.14-76.67)=78.98$$

Then, by formula **2**, Art. **58**,

$Q = .54 \times 2.5 \times 78.98 = 107$ cu. ft. per sec. Ans.

60. McMath Formula.—The formula derived by McMath for conditions in St. Louis, Missouri, is similar to the Bürkli-Ziegler formula, but has a different index of the root in the radical part. The formulas for the flow per acre and the total flow are, respectively,

$$F = C_1 i \sqrt{\frac{S}{A}} \qquad (1)$$

and

$$Q = F A = C_1 i \sqrt[4]{S A^4} \qquad (2)$$

in which all the letters have the same meanings as in Art. **58**.

In St. Louis, sewers whose capacities are less than required by McMath's formula in which $C_1 = .75$ and $i = 2.75$ are known to be overtaxed.

EXAMPLES FOR PRACTICE

1. Find by the Bürkli-Ziegler formula, using Table VIII, the maximum flow in cubic feet per second from an area of 60 acres, for which C_1 is .75 and i is 2.1 inches per hour; the ground slopes at the rate of 1.8 feet per hundred feet. Ans. 69 cu. ft. per sec.

2. What is the maximum flow from a district for which $C_1 = .60$ and $i = 2$ inches per hour, if the area is 50 acres and the ground slopes at the rate of 1.5 feet per hundred feet? Ans. 44 cu. ft. per sec.

3. Solve example 1 by the McMath formula. Ans. 74 cu. ft. per sec.

SEWERAGE
(PART 2)

DESIGN OF SEWERS

INTRODUCTION

FORMS OF SEWER SECTIONS

1. Controlling Factors.—The circular section is the one most used for sewers, but there are many conditions under which other shapes are more economical to construct, or possess better hydraulic properties. Factors in the selection of the section may include variations in the amount of flow, construction difficulties, cost, character of foundation, head-room available, etc. The material of which the sewer is to be constructed may also affect the choice of section; for example, it is easier to construct a concrete sewer with a flat, or nearly flat, bottom.

2. Typical Sections.—Next to the circle, the most common form of sewer section is the egg-shaped section. Among the other shapes used more or less generally are the U-shape, the semielliptical, the horseshoe, the basket handle, the catenary, the parabolic or delta, the semicircular, and the rectangular. Of these, the semielliptical and horseshoe sections are probably used most.

3. Egg-Shaped and U-Shaped Sewers.—The flow in a storm-water or a combined sewer will necessarily fluctuate

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greatly. During a violent storm, the sewer may be taxed to its full capacity, whereas, during an extended drought, the flow may become very small. The sewer must be large enough to carry the greatest flow and, therefore, when the flow in a large circular sewer becomes very small, the cross-section of the flow will necessarily be shallow. Since the hydraulic mean radius for this condition is small, it follows that the velocity will be low; in many cases it may be insufficient to prevent the deposition of solid matter contained in the sewage. Hence, the circular form of section is not well adapted to sewers in which the flow of sewage varies considerably.

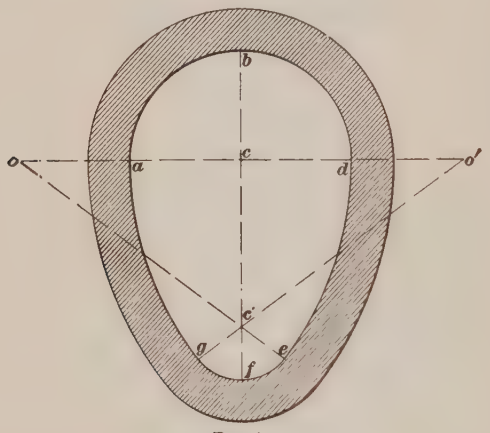

FIG. 1

The form of cross-section that provides the most satisfactory velocity for both large and small flows is the egg-shaped section, the general form of which is shown in Fig. 1. The part above the line *oo'*, called the *arch*, is a semicircle, and the part below the line *oo'* is formed by the three arcs *ag*, *ge*, and *ed*, the arcs *ag* and *ed* having equal radii. Hence, three different radii are used in constructing the section; namely *ca*, *cb*, or *cd* for the arch; *od*, *oe*, *o'a*, or *o'g* for the sides; and *c'g*, *c'f*, or *c'e* for the lower arc *gfe*, which is commonly called the *invert*.

The U-shaped section is used instead of the egg-shaped form in a few places. It may consist of a smaller section, usually

semicircular in shape, set into the bottom of a larger sewer, or it may have the shape of a true **U**.

4. Disadvantage of Inverts With Small Radius.—When a circular or an egg-shaped section is employed, the bottom of the trench in which the sewer is laid must be shaped to support the sharply curved bottom of the sewer. In soft soil a foundation must be provided to distribute the load more uniformly over the soil.

5. Semielliptical and Horseshoe Sections.—To overcome the difficulties in the construction of the invert of a circular or an egg-shaped section, the semielliptical section, shown in Fig. 2 (*a*), or the horseshoe section, shown in (*b*), may be used.

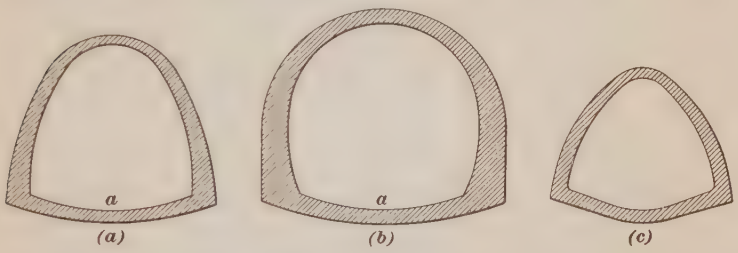

FIG. 2

These two forms are somewhat similar, but, in the horseshoe section, the bottom is flatter and the outside edges of the side walls are made vertical below the spring lines of the arch. Sometimes, three circular arcs are used instead of a semiellipse.

Semielliptical and horseshoe sections are not adapted to handling small flows, as the invert *a* is wide and shallow. For larger flows, however, the hydraulic properties are very good. Hence, for best results, the usual flow of sewage should not be less than one-third of the maximum. The shape is such that a large external load can be supported without aid from the backfilling. However, in soft soil, considerable stress is produced in the bottom, and, unless it is constructed of reinforced concrete, cracks will form at the center.

6. Unusual Sections.—The other forms of sections previously mentioned are rarely used. The basket-handle sec-

tion resembles somewhat the horseshoe, but is not used so much. The catenary shape lends itself well to tunnel work and is used mainly in that type of construction. The parabolic, or delta, section, shown in Fig. 2 (*c*), is economical to construct and handles relatively small flows well. The semicircular section was formerly used principally where headroom was lacking; it has been superseded largely by the rectangular section, which is cheaper to build and has better hydraulic properties.

MATERIALS FOR SEWERS

7. A sewer should be water-tight, both to prevent ground water from seeping in and to keep sewage from leaking out. The interior of the sewer should be smooth and uniform in cross-section, to facilitate the flow in the sewer. The material should be such that the cost of construction will be a minimum consistent with the required capacity, stability, and hydraulic properties.

Sewers may be built in place, the material being concrete, brick, stone, or other substances; or they may be constructed of pipes composed of vitrified clay, cement mortar, concrete, cast iron, steel, or wood. Most sewers up to 30 or 36 inches in diameter are built of vitrified-clay pipe with a circular section. Small concrete pipe is used to some extent for both circular and egg-shaped sections. The use of wood pipe in sewer work is limited almost entirely to outfalls. Cast-iron or steel pipe is often used for special purposes in sewer construction where strength and water-tightness are required.

Sewers larger than 36 inches in diameter are generally built in place in the trench. Reinforced concrete is the material predominately used in their construction. Up to about the year 1900, brick was a very popular material for sewers, and it is still used to a considerable extent, partly because under some conditions it is most economical and satisfactory. Large sewers may also be constructed of stone or vitrified-clay blocks.

FUNDAMENTAL PRINCIPLES OF SEWER DESIGN

8. Required Capacity of Sewers.—The most important problem in designing a sewerage system is to select the proper sizes of the various sewers. This question, which has already been studied in a general way, always should be investigated with reference to the particular case in hand. The capacity of a separate sewer must be sufficient to provide for the maximum rate of discharge. Since an allowance must be made for the ground water that may enter the sewer, it is usual to design separate sewers to flow half full when the maximum quantity of house sewage is being discharged.

The required capacity of a separate sewer may be much greater in a manufacturing than in a residence district, because the daily amount of sewage from a manufacturing plant may be discharged during a few hours, whereas that from a residence generally will be distributed throughout the greater part of the 24 hours. The waste of water is more nearly constant than its legitimate use. Consequently, the greater the percentage of water wasted, the more nearly uniform is the discharge of sewage, and the less does the maximum rate exceed the average.

The quantity of storm water to be carried away by a sewer is extremely uncertain and cannot be predicted with any degree of accuracy. However, it is usually considered preferable to allow a storm-water sewer to overflow occasionally than to have it entirely too large for ordinary conditions.

9. Formulas for Flow in Sewers.—The fundamental formulas used in designing any sewer are

$$v = c\sqrt{rs} \qquad (1)$$

and

$$Q = av \qquad (2)$$

in which v = velocity of flow, in feet per second;
c = coefficient;
r = hydraulic radius, in feet;
s = rate of grade of sewer;
Q = discharge of sewer, in cubic feet per second;
a = area, in square feet, of cross-section of flowing water or sewage.

The hydraulic radius is found by dividing the area of the cross-section of the flowing sewage by the wetted perimeter of the cross-section; both these quantities are determined from the shape of the sewer and the depth of flow. Sometimes the rate of grade is given or specified, but generally the elevations of controlling points fix its value, and it is computed by dividing the difference in elevation between the end points by the horizontal distance between these points. Strictly speaking, the inclined distance should be used, but since it is more convenient to measure the horizontal distance, and since the difference between the horizontal and inclined distances is extremely small, the horizontal distance is taken.

10. The value of c to be used in formula **1**, Art. **9**, is given by Kutter's formula, which may be written in either of the following forms:

$$c=\frac{\frac{1.811}{n}+41.65+\frac{.00281}{s}}{1+\frac{n}{\sqrt{r}}\left(41.65+\frac{.00281}{s}\right)} \qquad (1)$$

or

$$c=\frac{\frac{1}{n}+23+\frac{.00155}{s}}{.5521+\frac{n}{\sqrt{r}}\left(23+\frac{.00155}{s}\right)} \qquad (2)$$

Formula **2** is obtained from formula **1** by dividing both the numerator and the denominator by 1.811; the labor in the calculation of c is about the same in each case, but formula **1** is probably used oftener. The coefficient c depends to a considerable extent on the material of which the sewer is constructed, and allowance for this factor is made by introducing the number n, whose value has been determined by experiment for many different conditions. The values most generally used for sewers, and to be used in this Section, are: for vitrified-clay pipe and concrete, .013 when the work is well done and .015 elsewhere; for brick, .015. Values of c, corresponding to various values of the hydraulic radius and the

TABLE I

VALUES OF c FOR $n=.013$

Hydraulic Radius Feet	Grade				
	.0001 or 1 in 10,000	.0002 or 1 in 5,000	.0004 or 1 in 2,500	.001 or 1 in 1,000	.01 or 1 in 100
.1	54	59	62	65	66
.2	69	74	78	81	81
.3	78	83	87	89	90
.4	86	91	94	96	98
.6	96	100	103	104	106
.8	103	107	110	111	112
1.0	109	113	115	116	117
1.5	120	122	124	124	125
2.0	127	129	130	130	130
3.0	137	137	138	138	138
4.0	143	143	142	142	142
6.0	152	150	149	149	148
10.0	162	158	157	155	154

TABLE II

VALUES OF c FOR $n=.015$

Hydraulic Radius Feet	Grade				
	.0001 or 1 in 10,000	.0002 or 1 in 5,000	.0004 or 1 in 2,500	.001 or 1 in 1,000	.01 or 1 in 100
.1	44	48	50	54	54
.2	57	61	65	66	67
.3	65	69	73	74	76
.4	72	76	79	80	82
.6	81	85	87	88	90
.8	88	91	93	94	95
1.0	93	96	98	99	99
1.5	103	105	106	107	107
2.0	109	111	112	112	112
3.0	119	119	119	119	119
4.0	125	125	124	124	123
6.0	134	132	130	130	129
10.0	143	140	138	136	136

grade, are given in Table I for $n = .013$ and in Table II for $n = .015$.

In finding the value of c for a slope that is not listed in these tables, it is not necessary to interpolate between the columns. If the given slope is greater than .01, it is assumed to be .01; and if the given slope is less than .01, it is taken as the nearest value in the tables. For instance, from Table I, the value of c for a slope of .015 and a hydraulic radius of .4 foot is taken as 98, which is given in the column for $s = .01$; and the value of c for a slope of .0006 and a hydraulic radius of .6 foot is taken as 103, which is in the column for $s = .0004$. It is necessary, however, to interpolate for intermediate values of the hydraulic radius. Thus, if $n = .013$, $s = .0001$, and $r = .75$ foot, the value of c is found from Table I to be $96 + \frac{.75 - .6}{.8 - .6}(103 - 96)$, which equals 101. An average value of c is about 110 for $n = .013$ and 100 for $n = .015$.

11. Desirable Velocity in Sewers.—The minimum value of the velocity is taken as about 2 feet per second for ordinary sizes of separate sewers, and $2\frac{1}{2}$ or 3 feet per second for combined sewers. If the velocity is less than this amount, material in suspension is likely to settle and form deposits in the sewer. A mean velocity of at least 3 feet per second is preferable in all cases.

The velocity of flow should not be so great that the sewer eventually will be destroyed by erosion. For concrete sewers the maximum allowable velocity is about 8 feet per second. Even good brickwork will be affected by velocities exceeding 10 or 12 feet per second where the sewage contains sand. Vitrified clay pipe is very durable and it is seldom necessary to limit the velocity on account of erosion when this material is used. Although it will sometimes be impossible to reduce the velocity, it is desirable to set the limit at 8 to 12 feet per second according to the nature of the sewage and the cost of construction.

12. Necessary Refinement.—Extreme refinement in sewer computations is unnecessary. For one thing, the discharging

capacity of a sewer cannot be foretold accurately; the computed capacity may not be within 10 per cent. of the actual capacity, owing to the uncertainty in the number and effect of Y branches and manholes, the manner of laying the sewer, and other factors the influence of which cannot be estimated exactly. Furthermore, in pipe sewers it is seldom practical to provide the exact computed capacity for the reason that sewer pipes are manufactured only in certain diameters. Even circular and egg-shaped sections of brick and concrete are built in definite sizes. It is, therefore, customary to adopt the standard size whose capacity is just larger than that theoretically necessary. Besides, the probable growth of the territory, which must be taken into consideration, introduces a factor depending on sound judgment rather than on extreme refinement in mathematical work.

DESIGN OF CIRCULAR SEWERS

13. Standard Sizes of Sewer Pipes.—Vitrified pipes are made with regular and standard dimensions, which are common to all pipe factories throughout the United States. The pipes carried in stock have diameters of 4, 6, 8, 10, 12, 15, 18, 20, 21, and 24 inches. Some factories make at an additional cost special intermediate sizes. Larger pipes, which are 27, 30, and 36 inches in diameter, can also be obtained. The last-mentioned size is the largest usually made of vitrified clay. Concrete pipes are manufactured in various sizes from 4 inches to 72 inches or more.

14. Formulas for Design.—As already explained, separate sewers are designed to flow half full, and storm-water sewers to flow full.

Let d = diameter of circular sewer, in feet;
r = hydraulic radius, in feet;
a = area, in square feet, of cross-section of flowing sewage;
v = velocity of flow, in feet per second;
Q = discharge, in cubic feet per second;
s = rate of grade;
c = coefficient determined by Kutter's formula.

For a circular sewer flowing either full or half full, the hydraulic radius is equal to one-fourth of the diameter; that is,

$$r = \frac{d}{4} \qquad (1)$$

For a circular sewer flowing full, $a = \frac{\pi d^2}{4}$; and by formula **2**, Art. **9**,

$$Q = av = \frac{\pi d^2}{4} v$$

whence

$$v = \frac{4Q}{\pi d^2} \qquad (2)$$

If this value is substituted for v, and $\frac{d}{4}$ for r, in formula **1**, Art. **9**, the result is

$$\frac{4Q}{\pi d^2} = c\sqrt{\frac{ds}{4}}$$

from which

$$d = \sqrt[5]{\frac{6.48}{s}\left(\frac{Q}{c}\right)^2} \qquad (3)$$

For a circular sewer flowing half full, $a = \frac{1}{2} \times \frac{\pi d^2}{4} = \frac{\pi d^2}{8}$; and, in this case,

$$Q = \frac{\pi d^2}{8} v$$

whence

$$v = \frac{8Q}{\pi d^2} \qquad (4)$$

When the values of v and r are substituted in formula **1**, Art. **9**, it becomes

$$\frac{8Q}{\pi d^2} = c\sqrt{\frac{ds}{4}}$$

from which

$$d = \sqrt[5]{\frac{25.9}{s}\left(\frac{Q}{c}\right)^2} \qquad (5)$$

When the velocity and the hydraulic radius are given, the grade can be determined by the formula

$$s = \frac{v^2}{c^2 r} \qquad (6)$$

15. Applications of Formulas.—In all sewer problems to be solved by the use of formulas, two of the four quantities, d, Q, s, and v, are given and it is required to find the others. The most usual problems appear in one of the following classes:

1. Given the discharge Q and the grade s; to determine the diameter d and the velocity v.

2. Given the discharge Q and the diameter d; to determine the grade s and the velocity v.

3. Given the discharge Q and the velocity v: to determine the diameter d and the grade s.

4. Given the diameter d and the grade s; to determine the velocity v and the discharge Q.

EXAMPLE 1.—A concrete pipe sewer 30 inches in diameter is laid on a grade of .00125. Determine the velocity and the discharge when the pipe is flowing full.

SOLUTION.—Here $n = .013$, $s = .00125$, $d = 30$ in. $= 2.5$ ft., and $r = \frac{d}{4} = \frac{2.5}{4}$ $= .625$ ft. The value of c is found by interpolating in Table I, the column for a slope of .001 being used. Thus, $c = 104 + \frac{.625 - .6}{.8 - .6}(111 - 104) = 105$. Then by formula **1**, Art. **9**,

$$v = c\sqrt{rs} = 105\sqrt{.625 \times .00125} = 2.94 \text{ ft. per sec.} \quad \text{Ans.}$$

By formula **2**, Art. **9**,

$$Q = av = .7854 \times 2.5^2 \times 2.94 = 14.5 \text{ cu. ft. per sec.} \quad \text{Ans.}$$

EXAMPLE 2.—A vitrified-pipe sewer in a separate system is to discharge 4 cubic feet per second, the grade being fixed at .0016. Compute the diameter and the velocity of flow.

SOLUTION.—Since separate sewers are designed to flow half full, formula **5**, Art. **14**, applies to this example. The value of c cannot be computed yet because the diameter, and, consequently, the hydraulic radius, is not known. Hence, c is assumed as 110 and a trial value of d is determined. Thus,

$$d = \sqrt[5]{\frac{25.9}{s}\left(\frac{Q}{c}\right)^2} = \sqrt[5]{\frac{25.9}{.0016}\left(\frac{4}{110}\right)^2} = 1.85 \text{ ft.}$$

The next larger standard size, which is 24 in., or 2 ft., will be tried. For this diameter, the hydraulic radius is $\frac{2}{4}$ or .5 ft.; and from Table I, for r

$= .5$ and $s = .001$, the value of c is 100. When this value of c is substituted in the formula for d, the result is

$$d = \sqrt[8]{\frac{25.9}{.0016}\left(\frac{4}{100}\right)^2} = 1.92 \text{ ft.}$$

A 24-in. pipe will, therefore, be used. Ans.

By formula **1**, Art. **9**,

$$v = c\sqrt{rs} = 100\sqrt{.5 \times .0016} = 2.83 \text{ ft. per sec.} \quad \text{Ans.}$$

EXAMPLE 3.—A 48-inch circular brick sewer is to discharge 100 cubic feet per second when flowing full. Compute the required grade.

SOLUTION.—By formula **2**, Art. **14**,

$$v = \frac{4Q}{\pi d^2} = \frac{4 \times 100}{3.1416 \times 4^2} = 7.96 \text{ ft. per sec.}$$

To apply formula **6**, Art. **14**, it is necessary to know c, which depends on s. Here, $r = \frac{d}{4} = \frac{4}{4} = 1$ ft.; and, from Table II, the average value of c for this hydraulic radius is 96. Hence, as a trial,

$$s = \frac{v^2}{c^2 r} = \frac{7.96^2}{96^2 \times 1} = .00688$$

For this value of s, c is 99. Therefore. the grade must be computed again. It is

$$s = \frac{7.96^2}{99^2 \times 1} = .00646. \quad \text{Ans.}$$

16. Tables for Circular Sewers.—In Tables III and IV are given the velocities and discharges, when flowing full, for circular sewers of different diameters laid on various grades. Table III is for vitrified-clay and concrete pipe, for which n is .013; and Table IV is for brick and concrete sewers, for which n is .015. Not all the columns are full. For instance, in Table III no velocity or discharge is given for a 24-inch diameter and a grade of .03. The reason is that it is not desirable to use a sewer of this size on such a grade, because the velocity would be more than the usual allowable maximum rate of flow.

EXAMPLE 1.—Solve example **1**, Art. **15**, by the use of Table III.

SOLUTION.—For a diameter of 30 in. and a grade of .0010, the values in the table are $v = 2.65$ and $Q = 13.0$; for a grade of .0015, $v = 3.28$ and

TABLE III

VELOCITY AND DISCHARGE FOR CIRCULAR PIPE SEWERS OF VITRIFIED CLAY OR SMOOTH CONCRETE, FLOWING FULL

(n=.013; Q in cubic feet per second; v in feet per second)

Diameter Inches	Grade .05		Grade .04		Grade .03		Grade .025		Grade .0200		Grade .0175	
	v	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*
6	5.62	1.10	5.02	.986	4.35	.854	3.97	.780	3.55	.697	3.32	.652
8	7.13	2.49	6.38	2.23	5.52	1.93	5.04	1.76	4.51	1.57	4.22	1.47
10	8.46	4.61	7.57	4.13	6.56	3.58	5.99	3.27	5.35	2.92	5.01	2.73
12	9.73	7.64	8.70	6.83	7.53	5.91	6.88	5.40	6.15	4.83	5.75	4.52
15			10.3	12.6	8.91	10.9	8.13	9.98	7.27	8.92	6.80	8.34
18					10.2	18.0	9.30	16.4	8.31	14.7	7.78	13.7
20							10.0	21.8	8.95	19.5	8.37	18.3
21							10.4	25.0	9.27	22.3	8.67	20.9
24									10.2	32.0	9.54	30.0
27											10.4	41.4
30												
36												

Diameter Inches	Grade .0150		Grade .0125		Grade .0100		Grade .0075		Grade .0050		Grade .0030	
	v	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*
6	3.07	.603	2.80	.550	2.51	.493	2.17	.426	1.78	.350	1.38	.271
8	3.90	1.36	3.56	1.24	3.19	1.11	2.76	.964	2.23	.778	1.72	.600
10	4.64	2.53	4.23	2.31	3.79	2.07	3.28	1.79	2.64	1.44	2.05	1.12
12	5.33	4.19	4.86	3.82	4.35	3.42	3.77	2.96	3.08	2.42	2.36	1.85
15	6.30	7.73	5.75	7.06	5.14	6.31	4.45	5.46	3.64	4.47	2.79	3.42
18	7.21	12.7	6.57	11.6	5.88	10.4	5.09	8.99	4.16	7.35	3.22	5.69
20	7.75	16.9	7.08	15.4	6.33	13.8	5.48	12.0	4.48	9.78	3.47	7.57
21	8.02	19.3	7.33	17.6	6.55	15.8	5.67	13.6	4.63	11.1	3.59	8.63
24	8.83	27.7	8.06	25.3	7.21	22.7	6.25	19.6	5.10	16.0	3.95	12.4
27	9.65	38.4	8.81	35.0	7.88	31.3	6.82	27.1	5.57	22.1	4.32	17.2
30	10.4	51.1	9.46	46.4	8.46	41.5	7.33	36.0	5.98	29.4	4.63	22.7
36			10.7	75.6	9.61	67.9	8.33	58.9	6.80	48.1	5.27	37.3

Diameter Inches	Grade .0020		Grade .0015		Grade .0010		Grade .00075		Grade .00050		Grade .00025	
	v	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*
6	1.12	.220	.959	.188								
8	1.41	.492	1.22	.426	.982	.343						
10	1.67	.911	1.45	.791	1.17	.638	1.01	.552	.816	.445		
12	1.92	1.51	1.67	1.31	1.34	1.05	1.16	.911	.939	.737		
15	2.28	2.80	1.97	2.42	1.59	1.95	1.38	1.69	1.11	1.36	.760	.933
18	2.60	4.59	2.25	3.98	1.84	3.25	1.58	2.79	1.27	2.24	.881	1.56
20	2.83	6.18	2.43	5.30	1.98	4.32	1.70	3.71	1.39	3.03	.950	2.07
21	2.93	7.05	2.51	6.04	2.05	4.93	1.78	4.28	1.44	3.46	.984	2.37
24	3.23	10.1	2.79	8.77	2.26	7.10	1.96	6.16	1.58	4.96	1.09	3.43
27	3.49	13.9	3.02	12.0	2.47	9.82	2.12	8.43	1.73	6.88	1.19	4.73
30	3.78	18.6	3.28	16.1	2.65	13.0	2.30	11.3	1.86	9.13	1.29	6.33
36	4.30	30.4	3.72	26.3	3.01	21.3	2.61	18.5	2.11	14.9	1.47	10.4

$Q=16.1$. Since the given grade is midway between .0010 and .0015, the required velocity is $\frac{2.65+3.28}{2}=2.96$ ft. per sec. Ans.

The required discharge is $\frac{13.0+16.1}{2}=14.5$ cu. ft. per sec. Ans.

EXAMPLE 2.—Solve example 2, Art. **15**, by the use of Table III.

SOLUTION.—In this case the discharge for a sewer flowing half full is to be 4 cu. ft. per sec.; but since the hydraulic radius, the slope, and the coefficient c have the same values as for a full sewer, the same results will be obtained as for a full sewer with a discharge of 2×4, or 8, cu. ft. per sec. In Table III, the grade just less than the given value, .0016, is .0015; and the discharge for this grade just less than the required amount, 8, is 6.04. The corresponding diameter is 21 in. However, since the discharge from a 21-in. pipe on a grade of .0020 is also less than 8, this diameter is not large enough. A 24-in. pipe can be used, as the discharge 8.77 on a grade of .0015 is greater than 8, and the discharge on a grade of .0016 will be even larger. Ans.

For a 24-in. pipe on a grade of .0015, the velocity is 2.79, and on a grade of .0020, it is 3.23. On a grade of .0016,

$$v=2.79+\frac{.0016-.0015}{.0020-.0015}(3.23-2.79)=2.88 \text{ ft. per sec.} \quad \text{Ans.}$$

As in this case, the velocity or discharge determined by the use of Table III may differ a little from the value computed by using Table I and the formulas of Art. **9**. However, slight differences may be disregarded, for the results are only approximate at best.

EXAMPLE 3.—Solve example 3, Art. **15**, by the use of Table IV.

SOLUTION.—From the table the discharge of a 48-in. sewer on a grade of .004 is 79.3 cu. ft. per sec., and on a grade of .007, it is 105. Hence, the required grade for a discharge of 100 cu. ft. per sec.. is

$$.004+\frac{100-79.3}{105-79.3}(.007-.004)=.00642. \quad \text{Ans.}$$

17. Diagrams for Circular Sewers.—Diagrams from which the required values can be obtained directly are often more convenient than tables, especially in sewer design where extreme accuracy is unnecessary. A form of diagram, prepared by G. S. Pierson for $n=.013$ and a full sewer, is shown in Fig. 3. Its use is illustrated in the following examples.

EXAMPLE 1.—Solve example 1, Art. **15**, by the use of Fig. 3.

SOLUTION.—The first step is to locate the point of intersection of the lines representing the values of the two given quantities. In this case, the

TABLE IV

VELOCITY AND DISCHARGE FOR CIRCULAR SEWERS OF BRICK OR CONCRETE, FLOWING FULL

($n = .015$; Q in cubic feet per second; v in feet per second)

Diameter Inches	Grade .01		Grade .007		Grade .004		Grade .002		Grade .001		Grade .00075		Grade .00050		Grade .00025	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
24	6.09	19.1	5.09	16.0	3.84	12.1	2.71	8.50	1.90	5.97	1.64	5.15	1.33	4.17	.915	2.87
30	7.16	35.2	5.99	29.4	4.52	22.2	3.19	15.6	2.24	11.0	1.93	9.49	1.57	7.69	1.08	5.32
36	8.16	57.7	6.82	48.2	5.15	36.4	3.63	25.7	2.56	18.1	2.21	15.6	1.79	12.6	1.24	8.77
42	9.10	87.5	7.61	73.2	5.75	55.3	4.05	39.0	2.85	27.4	2.46	23.7	2.00	19.2	1.39	13.4
48	9.99	126	8.35	105	6.31	79.3	4.45	55.9	3.13	39.4	2.71	34.0	2.20	27.6	1.53	19.2
54	10.8	172	9.06	144	6.84	109	4.83	76.8	3.40	54.1	2.94	46.7	2.39	38.0	1.67	26.5
60			9.73	191	7.35	144	5.19	102	3.66	71.8	3.16	62.1	2.57	50.5	1.80	35.3
66			10.4	247	7.84	186	5.54	132	3.90	92.8	3.37	80.2	2.75	65.2	1.92	45.6
72					8.32	235	5.87	166	4.14	117	3.58	101	2.91	82.4	2.04	57.7
78					8.77	291	6.20	206	4.37	145	3.78	125	3.08	102	2.16	71.6
84					9.22	355	6.51	251	4.59	177	3.97	153	3.24	125	2.27	87.4
90					9.65	426	6.82	301	4.81	213	4.16	184	3.39	150	2.38	105
96					10.1	506	7.11	357	5.02	252	4.34	218	3.54	178	2.49	125
102							7.40	420	5.22	296	4.52	257	3.68	209	2.59	147
108							7.68	489	5.42	345	4.69	299	3.83	243	2.69	171
114							7.95	564	5.62	398	4.86	345	3.97	281	2.79	198
120							8.22	646	5.81	456	5.03	395	4.10	322	2.89	227

diameter of the pipe is known to be 30 in. and the grade is .00125, which is equivalent to 100×.00125, or .125, ft. per 100 ft. Hence, it is required to find the intersection of the straight inclined line marked 30″ and an imaginary vertical line representing .125. This point of intersection lies almost exactly on the curved line marked 3, which indicates a velocity of 3 ft. per sec. Ans.

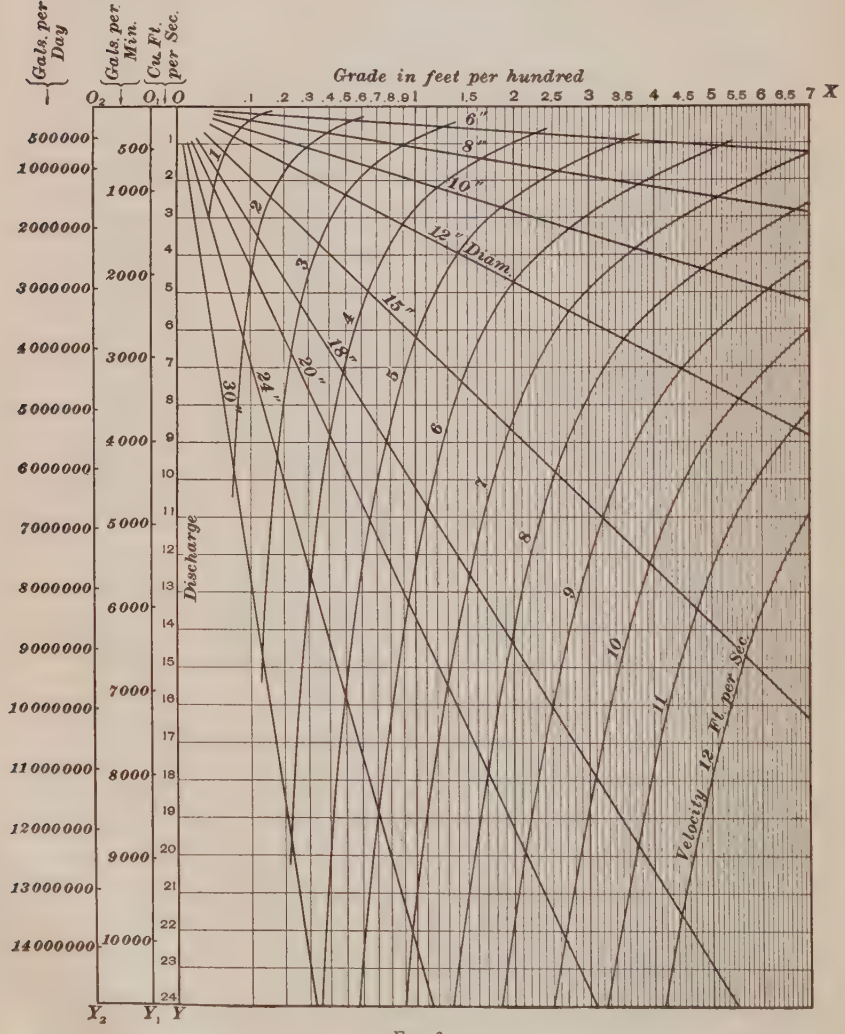

Fig. 3

The point of intersection also lies about midway between the horizontal lines marked 14 and 15 on the line OY. Therefore, the discharge is about 14.5 cu. ft. per sec. Ans.

EXAMPLE 2.—Solve example 2, Art. **15**, by the use of Fig. 3.

SOLUTION.—In order to use the diagram, the sewer will be assumed to flow full with a discharge of 2×4, or 8, cu. ft. per sec. The grade is .0016, or .16 ft. per 100 ft. Since the point of intersection of the horizontal line indicating a discharge of 8 cu. ft. per sec. with the vertical line representing a grade of .16 ft. per 100 ft. lies between the inclined lines for diameters of 20 and 24 in., the required diameter is the larger of these two values, or 24 in. Ans.

The point of intersection of the vertical line representing a grade of .16 ft. with the inclined line representing a diameter of 24 in. lies between the curves marked 2 and 3, and at a distance from the latter equal to about one-fifth of the entire distance. Hence, the required velocity is $3-\frac{1}{5}\times1=2.8$ ft. per sec. Ans.

18. Effect of Size of Sewer on Velocity.—When two circular sewers laid on the same grade are flowing full, the velocity is greater in the larger sewer than in the smaller one. On the other hand, if the flow is shallow in two sewers of different sizes carrying equal volumes of sewage, the velocity is greater in the smaller sewer than in the larger one. The reason is that the ratio of the depth of sewage to the diameter of the section, and the hydraulic radius of the section, are less in the larger sewer than in the smaller one. It is seen from Table I or II that the coefficient c decreases as the hydraulic radius becomes smaller; and, from formula **1**, Art. **9**, it is evident that the velocity diminishes quite rapidly as the depth of flow becomes lower, since both r and c decrease. Hence, it must not be assumed that the velocity in a sewer may be increased by the use of a larger pipe, the grade remaining the same. On the contrary, the velocity will be diminished, and the danger of deposits made greater, by increasing the size of the sewer.

19. Effect of Depth of Flow on Velocity and Discharge. The relative values of the area of sewage, the velocity, and the discharge for different depths of flow in a circular sewer are shown in Fig. 4, the value for the sewer flowing full being taken as unity in each case. The curve for area applies to

any circular sewer. The velocity, however, depends on the grade of the sewer and on the numerical value of the hydraulic radius. The velocity curve, therefore, is not exactly correct for all diameters and grades, but it is sufficiently accurate for ordinary purposes, provided the different depths relate to the same sewer. For any depth of flow, the value given by the discharge curve is equal to the product of the corresponding

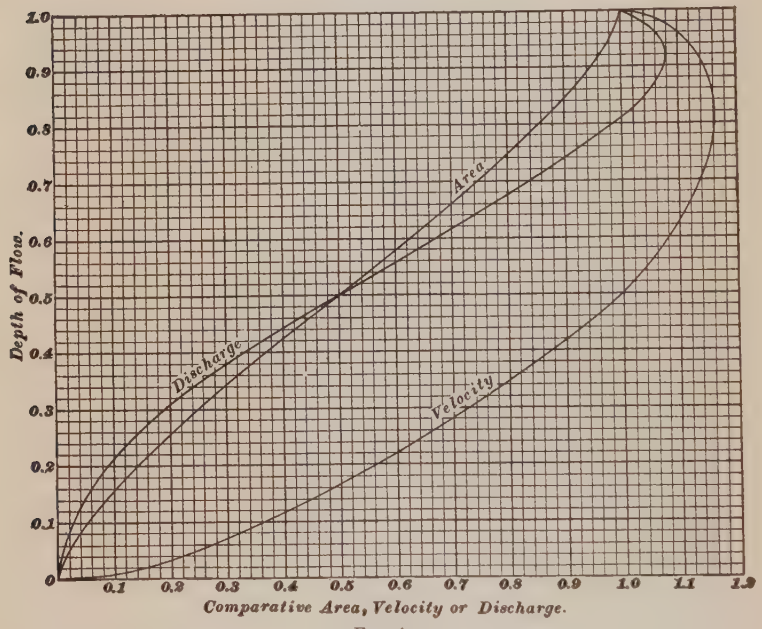

FIG. 4

values given by the area and velocity curves. For shallow depths, it is seen that the velocity diminishes very rapidly as the volume of sewage, and. consequently, the depth, becomes smaller.

The maximum velocity occurs when the depth of flow is about .8 of the diameter, and the maximum discharge takes place when the depth is about .93 of the diameter. The velocity is the same when the sewer flows half full as when it flows full, and the discharge is just half as great in the former case as in the latter.

Fig. 4 is convenient when the velocity or discharge in a sewer flowing full has been determined and it is necessary to find the velocity or discharge when the sewer is flowing partly full. Suppose, for example, that the velocity in a sewer flowing full is 5.6 feet per second and it is required to compute the velocity when the depth of flow is .8 of the diameter. Since, in Fig. 4, the abscissa of the velocity curve corresponding to the ordinate .8 is 1.16, the desired velocity equals $1.16 \times 5.6 = 6.5$ feet per second.

EXAMPLES FOR PRACTICE

1. An 8-inch pipe sewer flowing half full is laid on a grade of .006. Compute by the use of formulas (*a*) the velocity; and (*b*) the discharge in cubic feet per second.

Ans. { (*a*) 2.4 ft. per sec.
(*b*) .419 cu. ft. per sec.

2. What is the required diameter of a pipe sewer, flowing full, to discharge 3.6 cubic feet per second, the grade being .0025? Use formulas.

Ans. 18 in.

3. Determine by means of formulas the proper grade for a 36-inch circular brick sewer flowing full and discharging at the rate of 20 cubic feet per second.

Ans. .00125

4. Solve example 1 by the use of Table III.

Ans. { (*a*) 2.44 ft. per sec.
(*b*) .426 cu. ft. per sec.

5. Solve example 2 by the use of Fig. 3.

6. Solve example 3 by the use of Table IV.

Ans. .00125

DESIGN OF EGG-SHAPED SEWERS

20. Forms of Cross-Section.—As already explained, an egg-shaped sewer has a semicircle for the arch, a circular arc for the invert, and two circular arcs for the sides. There are two forms for the interior cross-section of an egg-shaped sewer. In the form shown in Fig. 5 (*a*), which was formerly used, the radius of the invert is one-half of the radius of the arch; that is $R_1 = \frac{1}{2}R$. In the form shown in (*b*), which is known as the *new form*, the radius of the invert is one-quarter of the radius of the arch; that is, $R_1 = \frac{1}{4}R$. This later form was

designed for use where the flow is small, because, with the same volume of sewage, a slightly greater depth of flow and a somewhat higher velocity are obtained.

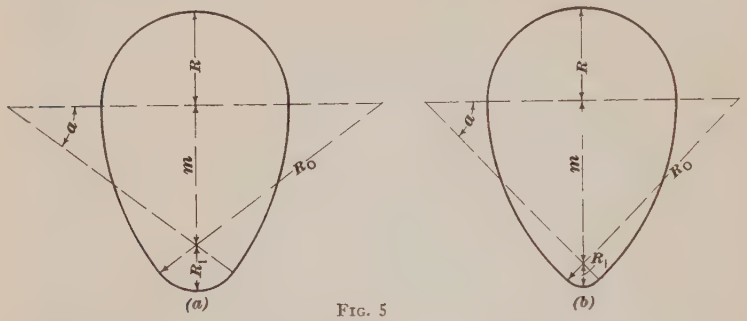

FIG. 5

21. Elements of Cross-Section.—The size of an egg-shaped sewer is specified by its greatest horizontal and vertical dimensions. The common widths are multiples of 6 inches, the smallest being 24 inches. Usually the height is $1\frac{1}{2}$ times the width. The relations among the dimensions shown in Fig. 5 are as follows:

$$m = 2R - R_1 \qquad (1)$$

$$R_0 = \tfrac{1}{2}\left(\frac{m^2}{R - R_1} + R + R_1\right) \qquad (2)$$

$$\sin a = \frac{m}{R_0 - R_1} \qquad (3)$$

Accurate values of the area and the wetted perimeter of an egg-shaped sewer cannot be computed easily from the dimensions of the sewer. However, when the sewer is flowing full, the following approximate formulas apply to both old and new forms closely enough for most practical purposes:

$$a = \frac{9}{2}R^2 \qquad (4)$$

$$p = \frac{63}{8}R \qquad (5)$$

$$r = \frac{a}{p} = \frac{4}{7}R \qquad (6)$$

in which a = area of section, in square feet;
R = radius of arch, in feet;
p = perimeter of section, in feet;
r = hydraulic radius, in feet.

22. Relative Capacities of Egg-Shaped and Circular Sewers.—Although the area and the hydraulic radius of an egg-shaped section flowing full can be computed approximately from the radius of the arch, it is generally best in designing an egg-shaped sewer to obtain first the required diameter of a circular sewer, and then to find the dimensions of an egg-shaped section having an equal capacity.

Let d = diameter of circular sewer;
d_1 = width of egg-shaped sewer having same area.

Then, for the old form, $d_1 = .8269d$ (1)

For the new form, $d_1 = .8393d$ (2)

For either form, the following approximate formula is close enough for most practical purposes.

$$d_1 = \tfrac{5}{6}d \qquad (3)$$

The hydraulic radius of an egg-shaped sewer is very nearly the same as the hydraulic radius for a circular sewer of equal area, and for practical purposes the two values may be considered to be equal. Hence, the discharges may be assumed to be equal when the areas are equal and the slope is the same.

EXAMPLE.—What must be the width of a concrete egg-shaped sewer which, when flowing full, is to discharge 100 cubic feet per second, the grade being .0015?

SOLUTION.—In this case, n is taken as .015, and the diameter of a circular sewer of the required capacity is found by the use of Table IV. For a 66-in. sewer on a grade of .0015, the discharge equals $92.8 + \frac{.0015-.0010}{.0020-.0010}(132-92.8) = 112$; for a 60-in. sewer, the discharge equals $71.8 + \frac{.0015-.0010}{.0020-.0010}(102-71.8) = 87$. Hence, the required diameter of a circular sewer is $60 + \frac{100-87}{112-87}(66-60)$, or 63.1 in. Then, by formula 3,

$$d_1 = \tfrac{5}{6}d = \tfrac{5}{6} \times 63.1 = 52.6 \text{ in.}$$

The next larger standard width of an egg-shaped sewer is 54 in. Ans.

23. Tables for Egg-Shaped Sewers.—In Table V are given the velocities and discharges, when flowing full, for different sizes of brick and concrete egg-shaped sewers laid on various grades.

EXAMPLE.—Solve the example in Art. 22 by the use of Table V.

SOLUTION.—From the table, the discharge for a 54 in.×81 in. sewer on a grade of .001 is 84.0 cu. ft. per sec., and on a grade of .002 is 119. Since the grade of .0015 is midway between .001 and .002, the discharge of a 54-in. sewer on the given grade is $\frac{84+119}{2}$ or 101.5. This is just greater than the required capacity of 100 and, therefore, 54 in. is the proper width. Ans.

EXAMPLES FOR PRACTICE

1. What will be the dimensions of an egg-shaped sewer whose area is the same as a circular sewer 36 inches in diameter? Ans. 30 in.×45 in.

2. Compute without Table V the required width of an egg-shaped sewer which, when flowing full, is to discharge 65 cubic feet per second, the grade being .0025. Ans. 42 in.

3. Solve example 2 by the use of Table V.

PRACTICAL APPLICATION OF DESIGN

24. In Fig. 6 is shown a section of a city for which storm sewers are to be designed. The maximum intensity of rainfall will be taken from Table II of *Sewerage*, Part 1, and the run-off will be computed by the rational method. It will be assumed that in every case the time required for the storm water to reach the sewer inlet from the most remote portions of the district will be 20 minutes, as the ground is fairly level. The coefficient of storm flow will be taken as .60 for all times of concentration and all districts. The routes of the sewers, which are indicated by broken lines, the locations of the manholes, which are indicated by circles and identified by the letters *M. H.* with a number, and the directions of flow, which are indicated by arrowheads, have been established, as shown in the figure. The elevation of the street surface at each manhole is also marked, because it is required that the top of

TABLE V

VELOCITY AND DISCHARGE FOR EGG-SHAPED SEWERS OF BRICK OR CONCRETE, FLOWING FULL

($n = .015$; Q in cubic feet per second; v in feet per second)

Size Inches	Grade .0100		Grade .0070		Grade .0040		Grade .0020		Grade .0010		Grade .0005		Grade .0003		Grade .0001	
	v	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*	*v*	*Q*
24× 36	6.69	29.8	5.59	25.0	4.22	18.8	2.98	13.3	2.09	9.33	1.46	6.51	1.11	4.97		
30× 45	7.86	54.8	6.57	45.8	4.96	34.6	3.50	24.4	2.46	17.1	1.72	12.0	1.32	9.16		
36× 54	8.94	89.7	7.48	75.0	5.65	56.7	3.98	40.0	2.80	28.1	1.96	19.7	1.50	15.1		
42× 63			8.33	114	6.29	85.9	4.44	60.6	3.12	42.7	2.19	29.9	1.68	22.9		
48× 72					6.90	123	4.87	86.8	3.43	61.2	2.41	42.9	1.85	33.0	1.03	18.3
54× 81							5.28	119	3.72	84.0	2.61	59.0	2.01	45.3	1.12	25.3
60× 90							5.67	158	4.00	111	2.81	78.3	2.16	60.2	1.21	33.7
66× 99							6.04	204	4.26	144	3.00	101	2.31	77.9	1.30	43.8
72×108							6.40	257	4.52	181	3.18	128	2.45	98.4	1.38	55.5
78×117							6.75	318	4.77	225	3.36	158	2.59	122	1.46	69.0
84×126									5.01	273	3.53	193	2.72	149	1.54	84.3
90×135									5.24	329	3.69	232	2.85	179	1.62	102

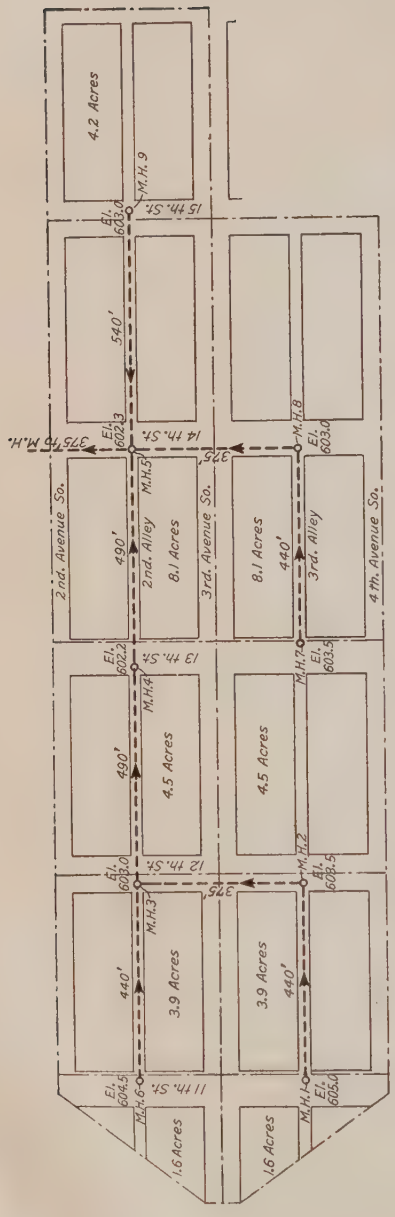

FIG. 6

the sewer should be at least 3 feet below the ground. For the purpose of this example it will be assumed that the districts tributary to the various manholes are as outlined by the dot-and-dash lines; the areas of these districts are written in the figure.

The data and the computations for determining the volume of storm-water run-off from each district and the sizes and slopes of the sewers may be conveniently arranged as in Table VI, which follows the form used by Metcalf and Eddy in *Sewerage and Sewage Disposal*. In column 1 of the table are numbers that identify the various sewer lines. The values in columns 2 to 6, inclusive, are taken directly from the map. Columns 2 and 3 show the numbers of the manholes between which the sewer in question is located; column 4 gives the name of the street or alley; column 5, the length of the sewer between manholes; and column 6, the area of the district immediately tributary to the section of the sewer under consideration.

The next step in designing the sewers is to compute the required capacity of each.

This is determined by applying the rational formula for storm-water run-off,

$$Q = CiA$$

in which Q = run-off, in cubic feet per second;
C = coefficient, assumed to be .60 in this example;
i = rate of rainfall, in inches per hour;
A = area of district, in acres.

The area A for each section of sewer is given in column 7 of the table. It is the total area tributary to the sewer in question, including both that immediately tributary and that tributary through other sewers. For example, the total area in line 2 is equal to the sum of the values in lines 1 and 2 of column 6, because the sewer between manholes 2 and 3 carries both the water from the district immediately tributary to manhole 2 and the water from the sewer between manholes 1 and 2; thus, $1.6+3.9=5.5$. Again, the total area in line 5 equals the sum of the values in lines 1, 2, 3, and 4; or $1.6+3.9+1.6+3.9=11.0$.

In order to determine the rate of rainfall to be used for each district, it is necessary to find first the proper time of concentration, that is, the time required for the water from the most remote points to reach the section of the sewer under consideration. This is given in column 8. For lines 1, 3, and 6, it is simply the time required for the water to reach the inlets to the sewers, which has been assumed to be 20 minutes. For the other lines, the time of concentration is equal to 20 minutes plus the time required for the water to flow through the sewers to the point in question. The time of flow in each section of sewer between manholes is given in column 9. It is first found in each case by assuming a velocity of flow in the sewer of 3 feet per second or 180 feet per minute, and dividing the length of the sewer by 180. If the actual velocity of flow as afterwards computed is not 3 feet per second, the time must be corrected. For line 5 the assumed time of concentration given in column 8 equals 20 minutes plus the sum of the values in lines 1, 2, and 4 of column 9; thus, $20+2.4+2.0+2.7=27.1$. For line 9, the time equals 20 plus the sum for

lines 1, 2, 4, and 5, as this is the longest route through sewers; it equals $20+2.4+2.0+2.7+2.7=29.8$.

The rate of rainfall corresponding to a given duration of storm is found from Table II of *Sewerage*, Part 1. The values for the various lines are given in column 10 of Table VI of this Section, the duration of storm being taken equal to the time of concentration in column 8. Now the required capacity of each sewer, which is given in column 11, is computed by taking the product of the values in columns 7 and 10, and multiplying the result by .60; for example, for line 5,

$$Q=CiA=.60\times2.99\times15.5=27.8$$

The next step is to determine the size and slope of each sewer for the capacity in column 11 and a velocity of about 3 feet per second. For sizes up to 36 inches in diameter, vitrified-clay pipe, for which $n=.013$, will be used. For larger sizes, concrete egg-shaped sewers, for which $n=.015$, will be used. Here, the computations for lines 2 and 9 only will be given.

Computations for Line 2.—For this line, Q is 11.1 cubic feet per second. Now the area a will be found for a velocity of 3 feet per second. From formula **2**, Art. **9**,

$$a=\frac{Q}{v}=\frac{11.1}{3}=3.7 \text{ square feet}$$

and

$$d=\sqrt{\frac{a}{.7854}}=\sqrt{\frac{3.7}{.7854}}=2.17 \text{ feet}=26 \text{ inches}$$

The next larger standard pipe, which is 27 inches in diameter, will be used. The required grade for a velocity of 3 feet per second is found from Table III, as follows:

$$s=.001+\frac{3-2.47}{3.02-2.47}(.0015-.001)=.00148$$

By formula **2**, Art. **9**, the capacity of the sewer is

$$Q=av=.7854\times2.25^2\times3=11.9 \text{ cubic feet per second}$$

Computations for Line 9.—Here, Q is 68.4 cubic feet per second; and from Table V it is seen that a 54 in.×81 in. egg-shaped sewer, laid on a grade of .0005, gives a velocity of 2.61 feet per second and a discharge of 59.0 cubic feet per

TABLE VI

VALUES FOR DESIGN OF STORM-WATER SEWERS

1	2	3	4	5	6	7	8	9	10
Line	Manhole at Upper End	Manhole at Lower End	Location	Length Feet	Area Adjacent to Inlet Acres	Total Area Tributary to Inlet Acres	Time of Concentration to Inlet Minutes	Time of Flow Through Sewer Minutes	Rate of Rainfall Inches per Hour
1	1	2	3rd Alley	440	1.6	1.6	20.	2.4	3.55
2	2	3	12th St.	375	3.9	5.5	22.4	2.0	3.35
3	6	3	2nd Alley	440	1.6	1.6	20.	2.4	3.55
4	3	4	2nd Alley	490	3.9	11.0	24.4	2.7	3.19
5	4	5	2nd Alley	490	4.5	15.5	27.1	2.7	2.99
6	7	8	3rd Alley	440	4.5	4.5	20.	2.4	3.55
7	8	5	14th St.	375	8.1	12.6	22.4	1.7	3.35
8	9	5	2nd Alley	540	4.2	4.2	20.	3.0	3.55
9	5	–	14th St.	375	8.1	40.4	29.8	2.0	2.82

1	11	12	13	14	15	16	17	18	19
Line	Required Capacity of Sewer Cubic Feet per Second	Size of Sewer Inches	Grade of Sewer	Velocity in Sewer Feet per Second	Estimated Capacity of Sewer Cubic Feet per Second	Ground Elevation Upper End	Fall of Sewer Feet	Elevation of Invert Upper End	Elevation of Invert Lower End
1	3.4	15	.00349	3.0	3.7	605.0	1.54	598.41	596.87
2	11.1	27	.00148	3.0	11.9	603.5	.56	596.87	596.31
3	3.4	15	.00349	3.0	3.7	604.5	1.54	598.10	596.56
4	21.1	36	.00099	3.0	21.2	603.0	.49	595.56	595.07
5	27.8	36×54	.00117	3.0	30.1	602.2	.57	594.07	593.50
6	9.6	24	.00181	3.1	9.7	603.5	.80	597.12	596.32
7	25.3	30×45	.00218	3.6	25.3	603.0	.82	595.07	594.25
8	8.9	24	.00174	3.0	9.4	603.0	.94	596.94	596.00
9	68.4	54×81	.00069	3.0	68.5	602.3	.26	592.50	592.24

second. The required grade and velocity to give a discharge of 68.4 may be found by interpolation. Thus, the grade equals

$$.0005+\frac{68.4-59.0}{84.0-59.0}(.0010-.0005)=.00069$$

and the velocity equals

$$2.61+\frac{68.4-59.0}{84.0-59.0}(3.72-2.61)=3.0 \text{ feet per second}$$

The sizes of the various sewers are given in column 12 of Table VI; the grades in column 13; the velocities in column 14; and the estimated capacities, as determined by the dimensions and grades, in column 15.

The last step is to establish the elevation of the invert at each manhole. The ground elevations given in column 16 are taken from the map, and the fall of the sewer shown in column 17 is computed by multiplying the length from column 5 by the grade from column 13. Thus, for line 4, the elevation of the ground at manhole 3 is 603.0 and the fall of the sewer between manholes 3 and 4 equals $490\times.00099=.49$ foot. Since the largest sewer leaves manhole 5 and the ground elevation is very low at that point, the controlling elevation of the invert will be there. The vertical dimension of the sewer is 81 inches, or 6.75 feet, and the top of the interior must be at least 3 feet below the ground. Hence, the required elevation of the invert at manhole 5 equals $602.3-6.75-3=592.55$, or, say, 592.50. Then, the elevation at the lower end in line 9 equals $592.50-.26=592.24$. Where a small pipe enters a manhole from which a larger sewer leaves, the invert at the smaller sewer is set somewhat above the bottom of the manhole to lessen the excavation for the pipe. The elevation of the lower end in line 5 is made 1 foot above that of the upper end in line 9, and the corresponding elevation of the upper end in line 5 is found by adding the fall in column 17. At manhole 5 the invert of the 24-inch pipe sewer is set 3.5 feet above the invert of the largest egg-shaped sewer.

The preceding example is only an incomplete illustration of the rational method of designing a storm-water sewer system.

Often practical limitations affect the grades and sizes, and it is necessary to make and study several possible plans, and to prepare estimates of the relative costs, in order to select the best layout.

CONSTRUCTION OF SEWERS

PIPE SEWERS

VITRIFIED-CLAY PIPE

25. Advantages.—Vitrified clay is the material most commonly used for small-sized sewers, as it has many advantages. It is reasonably cheap, and, since it can be made in many parts of the country, there are no excessive charges for its transportation. The surface is not only impervious but also is very durable and resistant to acids, steam, etc. The interior of a vitrified pipe is so smooth that a greater velocity is obtainable for the same grade than in a pipe of any other material. Vitrified clay is strong enough to resist the ordinary pressures from the surrounding earth and the backfill, even with heavy street loads on it. Although sewers do not usually carry sewage under pressure, vitrified pipe is able to resist a small internal pressure.

26. Standard Forms.—Vitrified sewer pipe is of the bell-and-spigot type, the common forms being shown in Fig. 7. In (*a*) is shown a straight pipe with the bell end at *b* and the spigot end at *s*; in (*b*), (*c*), and (*d*) are illustrated three curved forms for turning a right angle, an angle of 45 degrees, and an angle of $22\frac{1}{2}$ degrees, respectively; in (*e*) is shown a tee branch, or **T**, and in (*f*) a wye branch, or **Y**, which are used to make branch connections at angles of 90 degrees and 45 degrees, respectively. A **Y** or a **T** consists of an ordinary length of straight pipe intersected by a stub of the same or smaller diameter. The stub of a **Y** branch points toward the bell end of the straight part and makes an angle of about 45 degrees with the

straight part. A 45-degree curve is also called a *one-eighth curve*. Vitrified pipe is made in many other forms for use in plumbing work, but they are not found in street sewers.

27. Manufacture.—Vitrified pipe is made from various clays and shales with an admixture of other materials. The exact method of preparing the materials varies somewhat with their source and character, but the ingredients are always ground and mixed together. This part of the process is not unlike that used in making brick, the object being to have the

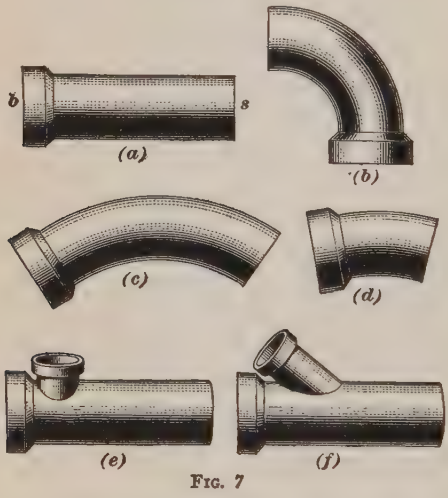

FIG. 7

ingredients thoroughly pulverized and well mixed. The mixture, which has the consistency of a stiff clayey mud, is shoveled or dropped from a conveyer into a pipe press, in which the pipe is formed. The lengths of pipe are allowed to dry for 2 weeks or so, and are then carefully moved to a brick kiln, shaped like a beehive and approximately 25 feet in diameter at the base. There a coal fire raises the temperature to about 2,500° Fahrenheit in about 7 days. This is the melting point of the aluminum silicate that makes up a large part of the clay and is also the temperature at which sodium chloride, or common salt, breaks up. Hence, if some salt is thrown into the kiln at this stage, the sodium that is set free combines with the

melted silicate on the surface of the pipe, and a glassy coating is formed there. The kiln is then allowed to cool for about a week, after which time the pipes are removed and are ready for use.

28. Length.—The lengths of pipes are 2, $2\frac{1}{2}$, and 3 feet, depending on the diameter. The greatest length is the most desirable, for the number of joints in the pipe line is reduced; but the length is limited to 3 feet, because damp clay will not stand up for a greater height.

29. Thickness and Strength.—The thickness of sewer pipe is based on experience in actual construction and on the results of a number of tests made on pipe of different sizes and makes. It is not dependent on the internal pressure, as is the case for water pipe. The general practice of factories is to make pipe of two thicknesses, one known as *standard pipe* and the other as *double-strength pipe*. Table VII shows the dimensions and weights that well-made standard pipe should have, according to the custom of the best factories, and Table VIII gives the values for double-strength pipe.

The strength of a pipe is a function of the thickness, the thicker pipe being able to carry a greater load without breaking. Tests indicate that standard pipe 12 inches in diameter or larger can carry a uniform load of about 2,000 pounds per lineal foot of pipe; and double-strength pipe, about 4,000 pounds. The load that sewer pipes must carry is the weight of the earth in the trench above them, with the additional weight of a motor truck or a steam roller, either of which may add from 1 to 2 tons per foot to the loading. A 12-inch pipe in an 8-foot trench will have to support $1 \times 8 \times 1$, or 8, cubic feet of earth, which weighs about 1,000 pounds, and a possible additional pressure of 4,000 pounds. Only a fraction of this loading, however, is transmitted to the pipe, the rest being supported by the sides of the trench. It is safer to use double-strength pipe when the depth of cover is less than twice the width of the trench, and when heavy surface loads may be expected. Under other conditions, standard pipe may be used.

TABLE VII

DIMENSIONS AND WEIGHTS OF STANDARD VITRIFIED PIPE

Pipes 8 inches and larger have deep scarified sockets

Inside Diameter Inches	Length of Pipe Feet	Thickness of Shell Inches	Depth of Socket Inches	Angular Space Inches	Measurement Over Socket Inches	Weight per Foot Pounds	Car Load Feet
4	2	9/16	1½	½	7	8	3,300
6	2	⅝	1¾	⅝	9½	14	2,000
8	2½	13/16	2½	⅝	12¼	24	1,200
10	2½	⅞	2¾	⅝	15½	30	900
12	2½	1	3	⅝	17	40	650
15	2½	1⅛	3	⅝	20½	60	450
18	2½	1¼	3½	⅝	24½	80	350
21	2½	1 7/16	3¾	⅝	28	110	240
24	2½	1⅝	4	⅝	32	140	200
27	3	2⅛	4	¾	37	215	120
30	3	2¼	4	¾	40	260	102
33	3	2⅜	5	1¼	44½	310	90
36	3	2½	5	1¼	48	360	75

TABLE VIII

DIMENSIONS AND WEIGHTS OF DOUBLE-STRENGTH VITRIFIED PIPE

All sizes have deep scarified sockets

Inside Diameter Inches	Length of Pipe Feet	Thickness of Shell Inches	Depth of Socket Inches	Weight per Foot Pounds	Car Load Feet
12	2½	1⅛	3	50	550
15	2½	1¼	3	70	400
18	2½	1½	3	100	300
21	2½	1¾	4	140	200
24	2½	2	4	180	150
27	3	2¼	4	240	111
30	3	2½	4	300	87
33	3	2⅝	5	340	78
36	3	2¾	5	390	69

30. Depth of Socket.—There are two types of sockets for vitrified sewer pipe, the *standard socket* and the *deep-and-wide*, or *deep, socket*. The depth referred to is the distance *d*, Fig. 8. The deep socket, as supplied by some factories, costs somewhat more than the standard socket, while other factories furnish both at the same price. The advantage of the deep socket lies in the fact that the material with which the space between the bell and the socket is filled can be rammed into the socket to a greater depth, and there is, therefore, less leakage through the joint. With the deep socket, there is also a wider annular space between the bell and the socket, and, consequently, a better joint is possible. The widths given in the fifth column of Table VII, which are specified by the American Society of Municipal Improvements, are recommended. However, some firms manufacture pipe with less annular space. At the same price, deep sockets should be used in preference to standard sockets; and they should always be used in wet ground.

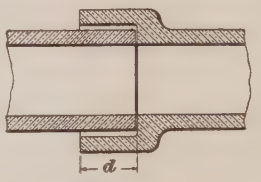

FIG. 8

31. Quality of Pipe.—In manufacturing sewer pipe, great care should be taken that the temperature is neither too low nor too high. If there is not sufficient heat, the surface is not thoroughly vitrified; in this case, a broken fragment shows an earthy, yellow section soft enough to be scratched by a knife, and, when a whole pipe is struck with a light hammer, a dull sound is heard instead of the clear ringing sound given by a good pipe. If the pipe is overburned, the clay contracts, especially at the ends of the pipe, and cracks are formed.

Factories are supposed to sell and ship only first-class pipe, unless an inferior grade is bought for the sake of reduced cost. All pipe should be carefully inspected before acceptance. The following points are to be investigated by an inspector:

1. Whether the thickness of the pipe is that called for.
2. Whether the depth of the socket and the width of the annular space between the spigot and the inside of the socket, when two pipes are fitted together, are those specified.

3. Whether any pipe intended to be straight is warped. If there is a variation of more than $\frac{1}{8}$ inch per foot of length in alinement, the pipe should be rejected.

4. Whether the inside diameter of the pipe varies more than is allowed by the specifications. For small pipes, this variation is usually about $\frac{1}{32}$ of the diameter; for larger pipes it should be less than $\frac{1}{32}$ of the diameter; even for very large pipes, such as 36-inch and 42-inch sizes, the variation should not exceed $\frac{3}{4}$ inch.

5. Whether there are any breaks or cracks passing through the pipe. There may be a single crack at either end of the pipe, provided it does not exceed 2 inches in length, or a single fracture in the socket, provided it does not exceed 3 inches in width nor 2 inches in length.

6. Whether there are any broken blisters on the inside of the pipe. If so, and if they are greater in thickness than $\frac{1}{8}$ inch, or larger in diameter than 3 inches, the pipe should be rejected.

7. Whether there are any unbroken blisters on the inside of the pipe. If so, unless they are less than $\frac{1}{4}$ inch in height and the pipe can be laid with the blisters on top, the pipe should be rejected.

8. Whether there are any fire-cracks or hair cracks sufficient to impair the strength or serviceability of the pipe. If so, the pipe should be rejected.

9. Whether the branches or spurs are firmly and thoroughly connected to the mains. If not, the pipe should be rejected.

10. Whether the pipe has a clear ringing sound or a dull earthy sound when struck with a hammer. In the former case, the pipe is probably good; in the latter, the pipe may be underburned or cracked, and should be rejected.

32. Cement Joints.—The joints in a sewer are the weakest part of the structure, because the line tends to settle at the joints, and as a result adjacent lengths of pipe may separate. The joints are not water-tight, both because the jointing material is pervious, and because laborers often do not pack the joint full. Therefore, more or less water comes

into the sewer in wet ground; this extra water takes up room, and, if pumping of the sewage is necessary, increases the cost. In dry ground, on the other hand, the liquid leaches out and pollutes the soil, leaving the solids behind to choke up the sewer.

The usual joint is made with oakum and cement, as shown in Fig. 9. An oakum packing, or *gasket*, which is laid around the spigot end as the latter enters the bell, is tamped back to the bottom of the joint. The rest of the space is then filled with cement mortar, mixed 1:1; enough mortar is provided at

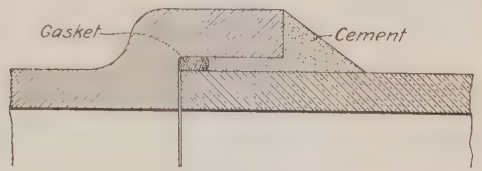

FIG. 9

each joint to have it extend out on the pipe, as shown. This cement should not be soft, but rather moist, so that it can be thoroughly tamped or rammed into the joint space. The cement outside the bell is best put on directly with the hands, which may be either bare or protected by rubber gloves.

Sometimes, pipes are joined without a gasket, the entire space being filled with cement. In this case, stiff mortar is spread in the socket on the inner surface of the lower part as the pipe lies in place in the trench. The spigot of the next length is then entered as high up as possible, pushed all the way back, and pressed down into the mortar. The remainder of the socket is filled after the new length is in place. This method is more rapid, but the pipes are less likely to be concentric, and there is more liability that the cement may get inside the pipe and form miniature dams against the flow.

33. Other Forms of Joints.—Cement is not an ideal material for joints. It cannot, except by the use of special forms, be run in, but must be placed by hand. Cement joints are often slighted when in difficult places, or under water, and at times part of the joint may be omitted. Furthermore, as cement

is brittle, it breaks in case of settlement, allowing leakage. Other materials are frequently used to insure a good joint. Pine tar and cement, and a mixture of sulphur and sand, have been used quite frequently; asphalt and linseed oil combinations are also popular. Some of the compounds are run in hot. There are also a number of patented joint compounds on the market. Nearly all the substitutes cost more than cement joints, and are used mainly in places where there is considerable ground water.

34. Construction of Pipe Sewers.—It is essential that the entire invert of a vitrified-pipe sewer be firmly supported in the trench. The excavation may be done without special care about to the center of the pipe, either a trenching machine or hand labor being employed; but the bottom of the trench should be shaped by men trained for the work. In good ground it should conform as closely as possible to the shape of the lower half of the pipe so that the pipe will be supported throughout its cylindrical part. A recess is dug in the trench bottom at each pipe length to receive the socket and to allow room for making the cement joint.

The pipe should be laid carefully with the socket toward the higher end of the line. For this work there need be only one man in the trench when the pipe is small and two men when the pipe is large. The joints should be cemented by one well-trained man. Earth should then be packed and tamped carefully in the recesses of the joints, and around the pipe. Special precautions should be taken under the lower half of the pipe and around branches. Filling and tamping should proceed equally on both sides of the pipe. The care required in refilling the remainder of the trench depends on circumstances.

A vitrified-pipe sewer as ordinarily laid is illustrated in Fig. 10. The excavation is being done by a trenching machine.

PIPES OF OTHER MATERIALS

35. Concrete Pipe.—Precast concrete or cement-mortar pipe sewers are used in many localities. Both circular and egg-shaped sections are on the market, in various sizes from 4 inches

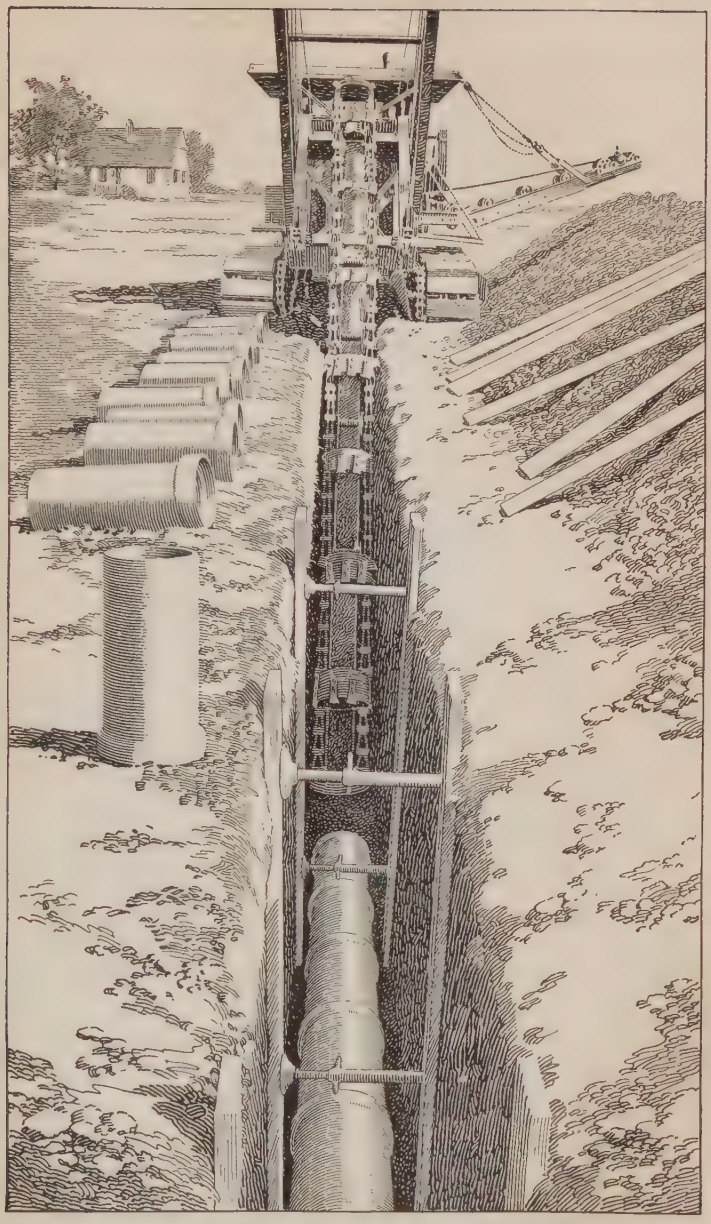

Fig. 10

to 72 inches or more. In sizes up to 10 inches, sand and cement are used without coarse aggregate in order to obtain a dense and uniform material. In the larger pipes, coarse aggregate not larger than one-half of the thickness of the pipe wall may be used, except where reinforcing steel is necessary, in which case the coarse aggregate should not be larger than one-quarter of the wall thickness. Reinforcing is always used in sizes larger than 36 inches and when conditions require it in sizes between 12 and 36 inches. To minimize the danger of breakage from settlement in soft earth, the base in the egg-shaped form is generally made flat.

In the smaller sizes, the walls of cement pipe are about 25 per cent. thicker than those of vitrified-clay pipe. Sometimes bevel or taper joints are provided, but in most cases the ordinary bell-and-spigot joint is used. Cement adheres very well to concrete pipes and makes a good jointing material, perhaps better than in vitrified pipe.

Owing to the inexpensive plant required, there are many small manufacturers of precast concrete pipe, not all of whom turn out consistently first-class work. Consequently, careful inspection of all concrete pipe sewers of this class is necessary, as poorly made pipe is not unusual.

36. Wooden Pipe.—Wooden pipe for sewers is used mainly where it is continually wet, as in outfalls and outlets, since wood is perishable and subject to short life under other conditions. There are two kinds of pipe, the machine-banded and the continuous-stave, the designs of which are similar to the types used for water lines. In sewer pipe, however, there is no internal pressure, and, therefore, only enough bands are required to hold the staves in place; they may be spaced 3 or 4 feet apart. The staves should be thin. In Fig. 11 is shown the cross-section of a wooden outfall sewer constructed in New London, Connecticut.

37. Iron and Steel Pipe.—Cast-iron pipes are often used for special purposes in sewer construction. They are required in many cases by railroads wherever the sewer passes under their tracks, in order to avoid the possibility of settlement and

breakage; they are also used under or over water-supply pipes where there is danger of contamination; and they have been used in places where the ground was saturated with water, and water-tight sewers were required. Wrought iron is not much used, owing to its short life under the conditions of use in sewers. In one city, however, where a brick sewer was found to leak excessively, a wrought-iron pipe was forced through inside of it, decreasing the diameter slightly, but making a tight sewer.

Steel pipe sewers are used quite extensively in some places, notably Jersey City, New Jersey. Steel pipe should be, and generally is, coated to prevent corrosion. The joints may be riveted, welded, or otherwise sealed. Both iron and steel sewers are more liable to damage from external causes than from internal pressures, and, therefore, should be set in concrete, or otherwise protected, where liable to damage.

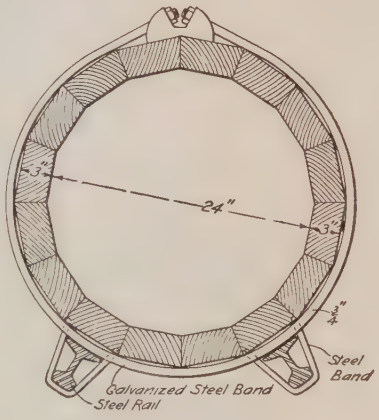

FIG. 11

SEWERS BUILT IN PLACE

CONCRETE SEWERS

38. Advantages of Large Concrete Sewers.—Concrete sewers built in place have an advantage over large sewers of brick because they are somewhat elastic. Steel reinforcement in the form of rods or wire netting is often imbedded in the concrete to aid in withstanding stresses due to settlement, poor workmanship, and other uncertain factors. Besides, concrete can be readily shaped to any desired form; it requires less skilled labor in construction; and is usually cheaper.

39. Construction Features.—Depending largely on the size, concrete sewers may be constructed in one, two, or more operations. Sewers up to 4 feet in diameter often may be

built in one operation at a considerable saving in cost, for, inasmuch as the quantity of concrete in a small sewer is not great, it is desirable to place as much concrete as possible in one operation. In this way, the expense of placing, handling, and removing the forms, and of placing the concrete mixer, chutes, and other equipment is reduced; and the cost of the concrete per cubic yard, in place, is also decreased. Furthermore, when more than one operation is required to

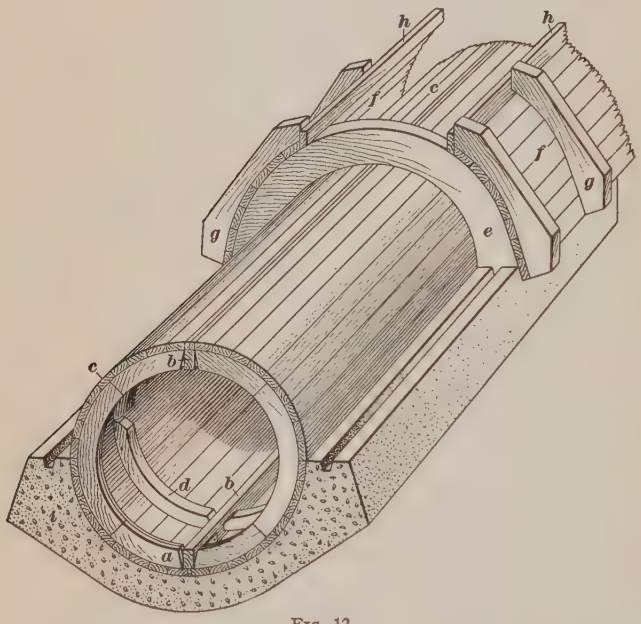

FIG. 12

complete the sewer section, it is necessary to leave open a considerable length of trench to allow the concrete in one part to set thoroughly before the next part is poured. Besides, extra care is required to obtain a tight joint between the two parts. In many places expansion joints of the tongue-and-groove variety are placed at intervals of 25 to 50 feet.

40. When a sewer is constructed in one operation, an entire set of forms is needed for the inner surface. In Fig. 12

is shown a form system for constructing a 3-foot sewer either in one operation or in two stages. In either method, an auxiliary frame consisting of the rings *a* and the stringers *b* is supported in the trench. Next, the inner forms, composed of the lagging *c* and the ribs *d*, are placed on this frame; the forms are built in four sections and are kept from collapsing by the stringers *b*. The bulkheads *e* are then set and the outer forms, consisting of the lagging *f* and the stiffeners *g*, are placed. There is no outer form between the splash boards *h*, the concrete being deposited through this opening and the surface being finished off without forms.

When the concrete has attained sufficient strength, the stringers *b* are withdrawn and the inner forms are removed. As the space within the rings *a* and the ribs *d* is entirely free, the forms that are removed can be passed ahead of those in place. If the concrete is poured in two operations, the invert *i* is cast first as shown, and the remainder of the section is poured afterwards. When the entire sewer is cast at once, the bulkhead is carried around the bottom also and is shaped to fit the trench bottom.

41. Large concrete sewers are always built in two or more stages. The invert is placed first, then the walls are constructed, and finally the arch is cast; or sometimes the lower part of the sewer is poured at one time and the upper part at another time. Grooved joints are commonly used for making a water-tight connection between two operations.

The construction of a large reinforced-concrete sewer in two stages is shown in Fig. 13. The invert is formed first by placing the concrete on the bottom of the trench and shaping it by means of templets in the form of the cross-section of the invert. Sometimes these templets are fastened securely in position, and the invert is shaped by a straightedge, which is held in contact with the templets and scrapes off any extra concrete. Another method of shaping the invert is to set longitudinal guide pieces about level with the horizontal axis of the sewer, and to slide along them a templet of the proper form. The invert is usually formed slightly larger than the

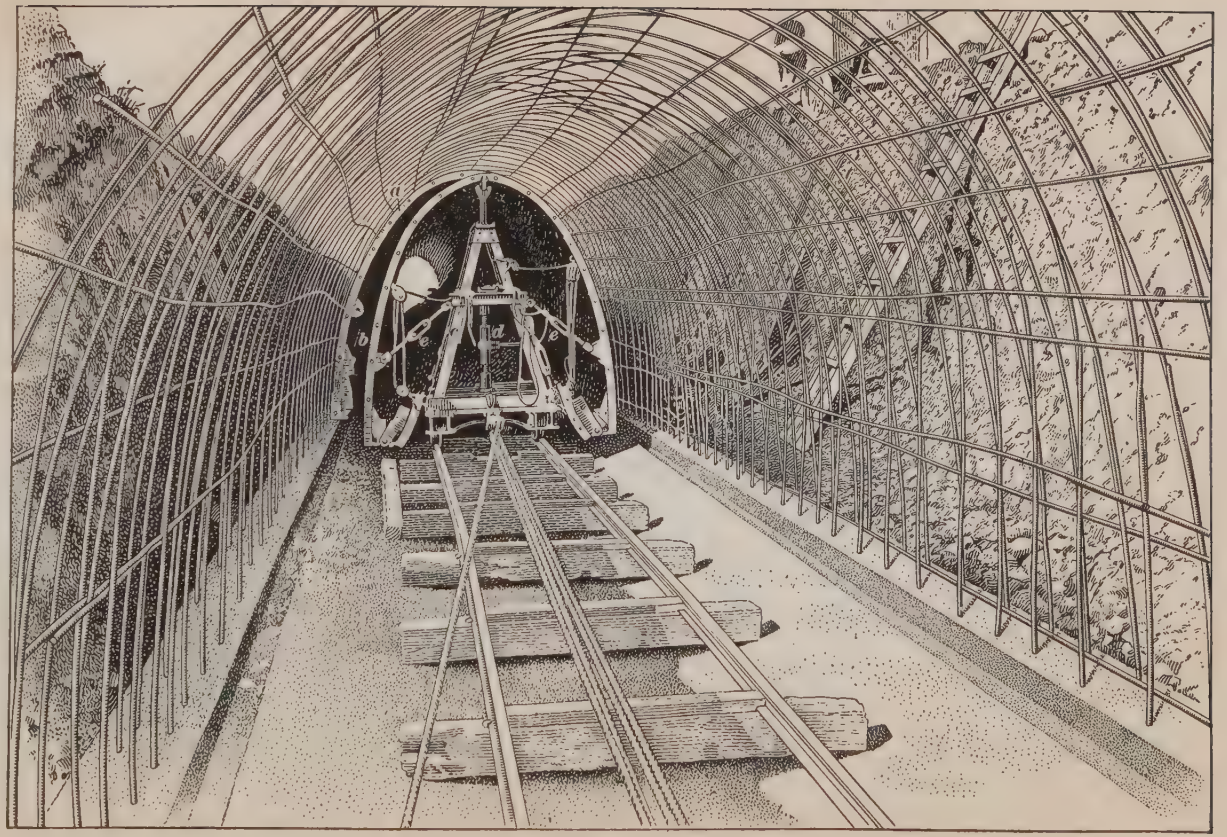

FIG. 13

finished sewer to allow for a coat of mortar, which should be applied and smoothed with a trowel before the concrete has taken its initial set. Where the velocity of flow in the sewer is expected to exceed about 8 feet per second, the invert may be lined with very hard-burned brick or paving brick; this lining should extend well up along the sides of the sewer.

When a sufficient length of the invert has been built, a track or conveying the forms for the walls and arch is laid on it.

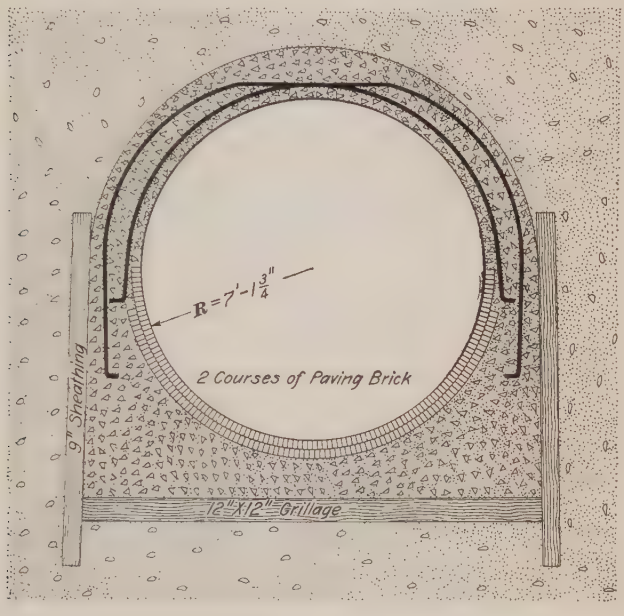

Fig. 14

In Fig. 13, the forms *a* are still in place while the forms *b* are being moved ahead from a section in which the concrete has already set. While being moved, the forms are mounted on the frame *c* by means of the jack *d* and the turnbuckles *e*. When they reach the proper location, they are adjusted in position by raising the jack and making the turnbuckles horizontal. The reinforcement is shown in place.

In Fig. 14, which is taken from the *Engineering Record*, is represented a circular reinforced-concrete sewer of large size.

The reinforcement in this sewer consists of bent rods, shown by the heavy lines, inserted at intervals of 15 inches, and also of longitudinal rods. The invert is lined with paving brick.

42. Forms.—The forms for concrete sewers must be tight, smooth, and thoroughly clean. They should also be low in first cost; durable, so that they can be used repeatedly; light in weight; and easy to handle, place, and move. Either wooden or metal forms may be used. Wooden forms are much used and are very satisfactory in many respects, but they cannot be used many times. After they have been used about ten times, they become so worn that it is cheaper to rebuild them or make new ones than to lose the time necessary for repairs and adjustments. Metal forms give very satisfactory service under most circumstances, and are durable. Local conditions will generally determine whether they will be cheaper than wooden forms. When wooden interior forms are used, they should be soaked thoroughly in water before being placed in the trench, because, if dry, they will extract water from the concrete and swell, and may crack the sewer for the entire length of the form. All forms should be rigidly supported so that they will not spring when the concrete is placed, and thus prevent a proper bond.

43. Quality of Concrete.—The concrete used for sewers should be of first-class quality, carefully proportioned to have as small a percentage of voids as possible. The concrete must be strong, to take up the stresses in the structure; and impervious, to keep ground water out of the sewers. A 1:2:4 mixture may be used for the arch, and a 1: $2\frac{1}{2}$: 5 mixture for the bottom. The mixing must be very thorough, and the concrete should be placed as soon as possible after being mixed. The tamping into place, especially around reinforcement, should be carefully done in order to obtain a smooth surface and an interior free from air-holes and pockets. As previously stated for reinforced-concrete pipe, the size of the largest aggregate should not exceed one-fourth of the thickness of a reinforced-concrete sewer wall. For sewer work, the

concrete should be so wet that a spade can be readily thrust into the mass to work it against the forms and around the reinforcing steel.

44. Reinforcement.—Transverse reinforcement is used in the walls of a concrete sewer to prevent caving. Longitudinal reinforcement to the extent of .2 to .4 per cent. of the area of the cross-section also is desirable to prevent cracks from shrinkage and from expansion and contraction due to temperature changes, as well as to carry the loads over weak places in the foundation.

It is usually difficult to place the reinforcement and to hold it in position. In general, the reinforcement should be wired and blocked in place, the blocks being removed as the concrete reaches them; or metal supports may be used. Where woven-wire reinforcement is used, the fabric hinders the flow of concrete, and extra care should be taken to work the concrete in and around the reinforcement.

45. Thickness of Concrete.—The thickness of the walls of a concrete sewer depends upon the size and shape of the sewer, the condition of the soil and backfill, etc. Mathematical

TABLE IX

THICKNESS OF CONCRETE SEWERS

Diameter of Sewer Feet	Thickness of Crown Inches	Thickness of Invert Inches	Thickness of Haunches Inches
3	4	5	10
6	7	8	17.5
9	10	11	25
12	13	14	32.5

analysis of the stresses in a sewer arch is possible, but extremely complicated. Several formulas have been proposed for determining the required thickness. W. B. Fuller's rule is to make the crown thickness, in inches, equal to the inside diameter, in feet, plus 1; the invert 1 inch thicker than the crown; and the haunches $2\frac{1}{2}$ times as thick as the crown. The values in

Table IX are derived by applying the rule. The minimum thicknesses are 4 inches for the crown, 5 inches for the invert, and 6 inches for the haunches.

The following formula is sometimes used for finding the crown thickness:

$$t = 3.36\sqrt{\frac{d}{2}} + 1.2$$

in which t = thickness of crown, in inches;
d = diameter of sewer, in feet.

The computed thicknesses must often be varied, however, according to the character of the soil and the depth of the trench. In wet, running soils, the lower part of the sewer may have to be from 2 to 4 times as thick as for ordinary conditions, and the thickness at the sides increased by a greater amount. In wide trenches more than 30 feet deep, arch thicknesses twice those given may be necessary.

BRICK SEWERS

46. Details of Construction.—In small sewers, all the bricks are generally laid with their greatest dimension lengthwise of the sewer, so as to make complete rings about 4 inches thick, which are keyed at the top of the arch. Another method of laying, which should be followed where possible, is to tie the two adjacent rings together by means of bricks laid as shown at *B*, Fig. 15. Bricks should never be laid in contact, but should always be bedded in mortar. In sewer work, it is best to lay them with what is called a *push joint*, or *shove joint;* the mortar is first spread thickly and the brick is them pushed into place until the mortar flushes out. Bricks should be wet before they are laid, as otherwise they will absorb moisture from the mortar and the latter will not set properly.

The rings in the invert are shaped to lines stretched between templets, called *profiles*, made for the number of rings required. The arch is laid on tight wooden forms, known as *centers*, which are withdrawn and advanced as soon as practicable. The outside of the arch is usually plastered with a coat of

cement mortar. The bricks are generally porous, but water is excluded and leakage prevented mainly by the mortar joints, called *collar joints*, between the separate rings and by the plastering on the exterior surface. The inside of the sewer should be cleaned and plastered smoothly, preferably with a rich mortar.

A change in the direction of a brick sewer is generally made by a long curve, and, as it is somewhat difficult to build a smooth curve of brick, the sewer is usually constructed in very short, straight sections. The form of the invert is tested by sliding a templet along it. It is customary to

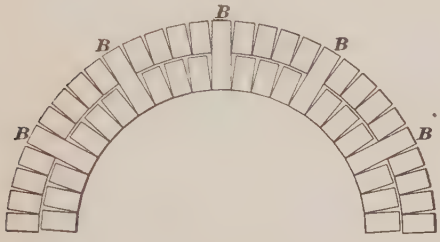

FIG. 15

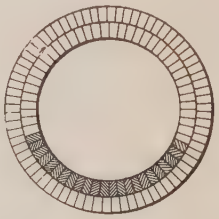

FIG. 16

increase the grade slightly around curves, and due allowance for this should be made in fixing the grades on the profiles.

47. Quality of Brick.—The inner ring of the invert of a brick sewer should be constructed of especially hard brick, either the hardest brick from the kiln or paving brick being used. The brick for the remainder of the sewer may be ordinary hard-burned building brick. In Fig. 16 is shown the portion of the sewer in which the special bricks are employed.

All the bricks should have small absorptive power. The inner surface of the sewer should be smooth, in order to promote the flow of sewage, and therefore well-formed bricks with smooth faces and sharp edges should be insisted on for the inner ring.

48. Thickness of Brick Sewers.—The thickness of a large brick sewer is largely a matter of judgment, though elaborate formulas have been developed to determine the prob-

able loads. Theoretically, the number of rings of brickwork, each 4 inches thick, depends on the span and on the loading; but the amount of the loading and the proportion of it that reaches the arch of the sewer are so uncertain, and the character of the soil in which the sewer is laid has so great an effect in holding the sewer together, that no close applications of theory are practical. When the conditions are not unusual, the following empirical formula will be found generally satisfactory for indicating the number of rings of brickwork required·

$$z = .4 + \frac{d(h-d)}{25}$$

in which z = number of 4-inch rings or courses;
d = internal diameter of a circular sewer, or horizontal diameter of an egg-shaped sewer in feet;
h = total depth of trench, in feet.

When the decimal in the value of z is less tnan .25, it may be disregarded; if it is .25 or more, the next larger whole number is taken.

Often, the character of the soil is such a controlling factor in determining the number of rings that the judgment of the designer is far more important than the results of the formula. For instance, in building a 24 in. × 36 in. egg-shaped sewer in quicksand in Massachusetts, it was found that one course of brick was not enough and the sewer had to be built 8 inches thick. On the other hand, in Indiana and Ohio, where the soil is a firm clay, sewers 6 feet in diameter have been built with but one ring.

In ordinary soil, sewers up to 30 inches in diameter may be made one brick thick. Sewers from 3 to 5 feet in diameter are generally made 2 bricks thick. When the diameter of the sewer is more than 5 feet, the span and loading become so great that a careful study of the conditions is necessary. The weight of the earth above is transmitted through the brickwork of the arch ring as a pressure between the separate bricks. This action is illustrated in Fig. 17, in which the forces P represent the pressure of the earth on the top of

the sewer, and the forces R represent the action of the sewer against the backing that is shoveled in between the sides of the trench and the sewer. If this backing is not firmly packed in, the sewer has a tendency to break, as shown at c. Owing to the uncertainty as to the action of this backing, no theory can be applied, unless the sewer is built to be self-sustaining

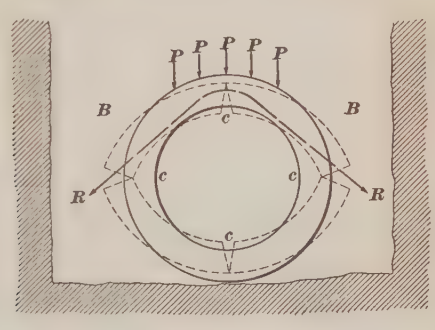

FIG. 17

and not to require any backing. It has been found that four rings are sufficient for the arch thickness of any large brick sewer. Just as in the case of concrete sewers, additional thickness is required at the haunches and in the invert.

SEGMENT BLOCK SEWERS

49. Sewers may also be constructed of specially shaped interlocking blocks of vitrified clay, called *segment blocks*. A perspective view of such a sewer is shown in Fig. 18. The faces of the blocks are curved to the arc of a circle. Block sewers are built in diameters from 30 to 108 inches. In some cases the blocks forming the invert of the sewer are so constructed that they serve as an underdrain for the sewer.

Among the advantages of segment-block sewers are a good velocity of flow, which is due to the nature of the material; cheapness, which is due in some measure to the large size of the blocks, the labor in laying them being less than for brick; and strength, which is due to the interlocking joints.

CONSTRUCTION LINES AND GRADES

50. Records of Works Previously Constructed.—Before a line for a sewer is staked out, it is necessary to ascertain the locations of all conduits, such as gas and water mains, that may have been laid in the street previously, in order that they may be avoided in the construction of the sewer. This is not always an easy matter. In fact, to obtain reliable information concerning the locations of the previously constructed works is often very difficult, and sometimes impossible, until they are

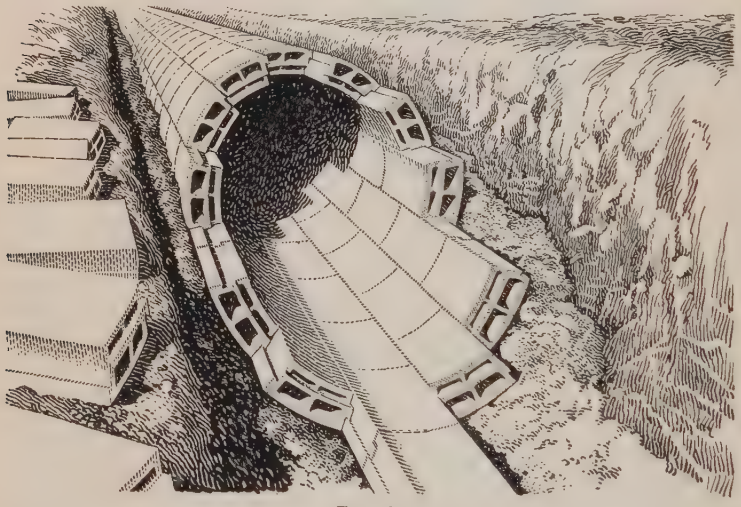

FIG. 18

met with in the excavations for the sewer. A map of these works should be obtained where possible; otherwise, a rough map or sketch should be made from such information as is obtainable.

51. Working Maps and Profiles.—When the preliminary investigations and surveys have been completed and the locations of all the sewers decided on, the next step is to prepare a working map for each street, showing the positions of all proposed main and lateral sewers and all accessories. This map is for convenient reference during the construction, and

need not be elaborate. Often it can be made from the preliminary map simply by making the necessary alterations and additions in red ink. If the changes are too numerous and important to permit of this, a tracing of the street may be made from the preliminary map, and on this tracing the sewer system may be drawn as finally located.

If the preliminary plans have been carefully drawn, the profiles should answer for construction work by correcting them from time to time as the work proceeds. It is generally unsafe to make very material changes in the preliminary profiles, particularly changes that would affect the grade elevation at

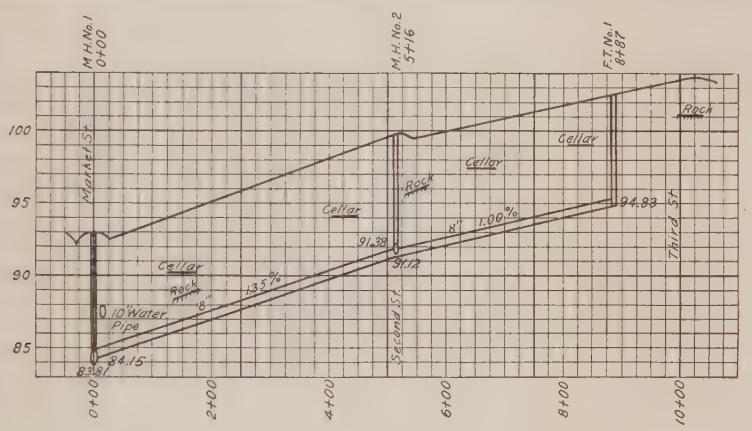

FIG. 19

points where various sewers or minor systems intersect. The assumption is that the elevations of these points were determined after a consideration of all the facts, some of which, at this stage of the work, may be lost sight of.

A convenient scale for a working map is 40 feet to the inch horizontal and 4 feet to the inch vertical. A map of this kind should show in detail the street-car lines, if any; fence and curb lines; trees, poles, fire hydrants, valve boxes, and gas drips; and the houses on either side of the street. The water, gas, and other conduit lines can be drawn in as accurately as possible, the best position for the sewer indicated, and the notes made out for the field party. The profile, which is best plotted on the

same sheet, should show all information from borings, the depths of rock and cellars, and the positions of all pipes and conduits encountered. Fig. 19 illustrates a profile for a sewer. Although the cross-sections of the various pipes are really circular, they are drawn as ovals because different scales are used for the vertical and horizontal distances.

52. Center Lines and Offset Lines.—After the working maps have been made, the center lines of the sewers, as established on these maps, should be laid out on the ground with the aid of a transit. The survey should start from the lower end, or outlet, of the principal trunk sewer and proceed upwards along it. Then the lines for the branch sewers should be run from their junctions with the main. For the purpose of checking the surveying work and of facilitating the construction of the final maps, the different lines should be tied together by cross-lines wherever convenient. The distance from the street lines to each sewer line thus run, and all distances along the sewer lines, should be measured with a steel tape and recorded.

It will be necessary to set a few temporary stakes along the center lines for transit hubs, and possibly for the purposes of taking measurements, but no stakes should be driven on the center line for permanent reference, as this line is within the limits of the excavation for the sewer and such stakes cannot be preserved. Stakes for use during construction are set on an offset line at a uniform distance from the center line. This offset distance should generally be about 2 feet greater than one-half of the width of the proposed trench. In order to avoid confusion, the offsets should, if possible, always be on the same side of the center line. The stakes along the offset line should be set at uniform intervals, 25 feet being a good distance.

The stakes should generally be about 1 inch square, with well-shaped tops, and their lengths should be such that they can be driven flush with the surface of the street without destroying the form of the top. Large spikes are often used conveniently instead of stakes, especially where the roadway is

hard. At transit stations, the point of measurement should be marked exactly by a tack in the top of the stake, but extreme accuracy is seldom required elsewhere.

Particular care should be taken to keep a permanent record of the station numbers for use during construction. High stakes, on which stations can be marked, cannot be kept very well near the trench. It is, therefore, good practice to set witness stakes at the curb line at intervals of 100 feet and to mark the station numbers on them.

53. Curves.—In the case of a large sewer, which can be cleaned by a man walking through it, a change in direction may be made by means of a curve, a circular curve generally being most convenient. Where the radius of the curve is short, as is commonly the case, the curve may usually be located most expeditiously as follows: The transit is set up over the point of intersection of the tangents, the angle between the tangents is bisected, and the center of the curve is located along this bisector. Then the arc of the curve is described from the center by swinging the tape. When the radius of the curve is comparatively long, points on the curve 25 feet apart may be located by deflection angles from the tangent or by chord deflections. Intermediate points on the curve can be set by stretching the tape between two 25-foot stations and measuring ordinates from this chord.

Where the change in direction is slight, an angle may be made in the line and the laying out of the curve left until the sewer is being constructed. The curve can then be located by offsets from points on the tangents, the offsets having been previously computed in the office.

54. Lines of Levels.—As soon as the sewer lines have been located on the ground, levels should be taken along them. The leveling generally can be done best by a party following the transit party closely. The elevations of the surface should be taken along the center line of the sewer at intervals of 25 feet, at all street intersections, and at all points where material changes occur in the inclination of the surface. The position

of the center line of the sewer can be obtained readily from the stakes set on the offset line by measuring with a leveling rod or by estimating. Intermediate stations may be located with sufficient accuracy by pacing from established points.

The levels should be checked carefully on each of the bench marks reached, and if these are not at convenient intervals for construction work, intermediate bench marks should be established. These readings on bench marks will not only check the sewer levels, but will serve as an additional check on the bench marks.

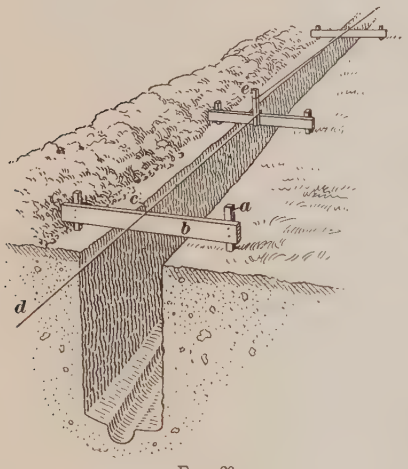

FIG. 20

55. Marking Line and Grade.—The best method of giving line and grade during construction is illustrated in Fig. 20. Heavy stakes *a* are driven in pairs, one on each side of the trench, and on them are marked points at equal distances above the grade line of the sewer. A board *b* with a straightedge uppermost and even with these marks, is then clamped or nailed on each pair of stakes. The boards should extend at least 18 inches beyond the edges of the trench and are best placed at 25-foot intervals. As many of the boards as possible should be at the same distance above the grade, so that a line between the tops of the boards will be parallel to the grade.

When the boards have been set at the proper heights, the center line of the trench is marked on each by a nail, as at *c*. A cord *d* is then stretched between the nails. The center line of the sewer is located by plumbing from this cord, and the invert is set by measuring down from the cord with a pole cut or marked to the proper length. Where two adjacent boards are at different distances above the grade of the sewer, the center line is marked on the lower one by means of an

upright, as at *e*, instead of a nail. The cord is then passed around the upright at the same height above grade as the higher board. Whether or not the grade boards are at the same distance above the sewer, grades can be given by the engineer before the trench is entirely excavated.

Where the ground is hard and stakes cannot be driven, the ends of the boards can be embedded firmly in piles of earth, the excavation having been started before the boards are set. Since it is difficult to set the boards to a definite grade in this way, the center line on each is usually marked by an upright on which the cord can be set at the proper elevation.

56. Need for Accuracy.—In order that the sewer may be in the exact position shown on the map of record, it should be constructed truly along the center line set by the engineer. The sewer should be built accurately to the established grade line so that the velocity and capacity will approximate the computed amounts. All grade elevations should be computed and set directly from bench marks, no dependence being placed on a depth of cut that may have been estimated from the levels taken in the preliminary or final survey. These estimated depths, however, serve well as approximate checks, and should be referred to frequently as the work proceeds. When the level has been set up to mark new grade elevations, a rod reading should be taken on the end of the invert completed, and the correctness of the computed reading for that point verified. This will check the previous work before the new is started.

DETAILS OF CONSTRUCTION

TRENCHING

57. General Considerations.—Sewers are generally constructed in open trenches. The simplest method of trenching is by hand labor, picks and shovels being used, but trenching is also done by means of steam shovels, mechanical excavators, cranes, and other apparatus. The method to be employed is

usually the one that is thought to be the cheapest for the conditions, and the choice depends upon the size of the job, the kind of soil, the depth of excavation, and other factors. Hand labor is generally most economical on small jobs, or where boulders, ledge rock, or other obstructions may interfere with the continuous operation of the machinery.

All excavated material should be thrown on one side of the trench, and as far from its edge as practicable. A space of at least 2 feet should be maintained between the edge of the trench and the excavated material to provide a pathway for the men and also to reduce the risk of material falling back into the trench.

Where the trench is more than 6 feet deep a laborer in the trench cannot throw the dirt far enough back from its edge and, therefore, another man must be provided to move the excavated material after it has been thrown out of the trench. When the trench is over 8 feet deep, a platform or stage must be built in the trench, and there must be a laborer at the bottom throwing dirt on the platform while another throws it out of the trench. For a trench more than 14 feet deep, two stages are necessary, and for greater depths, additional stages must be used.

For removing material from a deep trench, derricks and buckets, operated by hand, by horsepower, or by hoisting engines, are sometimes employed. Combined excavating and hoisting machines are often used, which, with three or four men, dig trenches up to 20 feet in depth and 5 feet in width. Conveying machinery is occasionally installed, by which dirt excavated at one point is carried back to a finished portion of the sewer to be used as backfilling.

58. The methods followed must be adapted to the work in hand, and must generally be somewhat varied in detail for each particular case. Each contractor usually has his own methods, which, to some extent, he prefers to follow; this he should, in justice, be allowed to do, so long as it does not in any way interfere with the quality and prompt execution of the work. But no questionable methods, such as might result in inferior

work, should be allowed under any consideration. The engineer should have personal charge of the construction, as matters will be arising continually, decisions on which cannot be entrusted safely to an inspector.

It is, in most cases, best to commence the construction at the lower end of the sewer and work toward the higher levels. This plan will permit the ground water to flow away through the constructed part of the sewer, or through a drain, and will keep the trench free from water; but no water should be allowed to run through the sewer until the cement has set. At least 24 hours should elapse after the sewer has been laid before water is let through it. Pipe sewers should always be laid with the sockets of the pipes toward the summit or higher end of the line; when the work proceeds upwards from the lower end, the spigot of each pipe is easily inserted in the socket of the pipe already laid.

In a few cases, however, where ground water is encountered in such quantities as to render the construction difficult, the work may be prosecuted more advantageously by working downwards, or toward the outlet. This method of procedure will permit the water to be drained away from the sewer into the lower part of the trench, and then pumped out. The part of the sewer under construction will be kept comparatively dry, which is in all cases desirable and is essential when the excavation is in certain kinds of material.

59. Bracing.—When the depth of a trench does not exceed 4 or 5 feet, the sides of the trench will usually stand without protection. In some soils, trenches of a much greater depth do not have to be protected, while in wet ground or in loose soils, like sand and gravel, even very shallow trenches need bracing to prevent caving. The element of time is also important in determining whether or not protection is required. A trench may stand for a few hours, but may fail in a day, especially when the banks are weakened by rain. Vibrations from passing vehicles or from blasting, or the load of excavated material usually piled alongside the trench, may hasten the action of caving.

It is difficult for a man inexperienced in earthwork to judge whether or not a trench should be braced. If not needed, the use of bracing is a waste of money; if needed, its lack is a source of danger to the men and the work. Although proper bracing of the trench is looked after principally by the contractor, because the risk of the undertaking falls on him, it is, nevertheless, a matter over which the engineer should keep a general supervision.

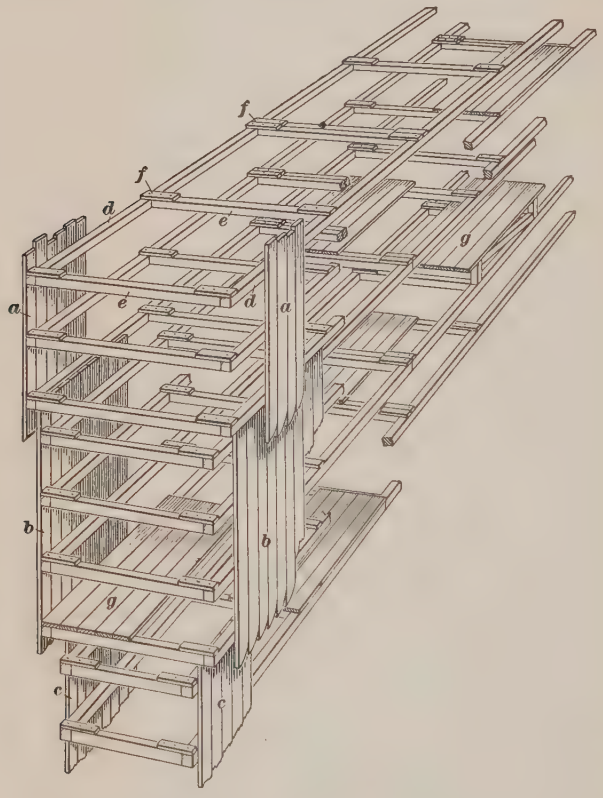

FIG. 21

60. Typical Bracing.—When support is needed, the banks of a sewer trench are commonly protected by means of a temporary framing of planks and timbers. A typical system is shown in Fig. 21. It consists essentially of rows of planks, *a*,

b, and *c*, known as *sheet piling*, or *sheeting*, which are driven vertically along each bank and are stiffened by horizontal timbers *d*, called *rangers* or *wales*, running lengthwise of the trench, and by braces *e* extending across the trench.

The sheeting may be about 1 inch thick, if sufficiently well supported; but it is more economical to use 2-inch plank, because the thicker material can be withdrawn and redriven

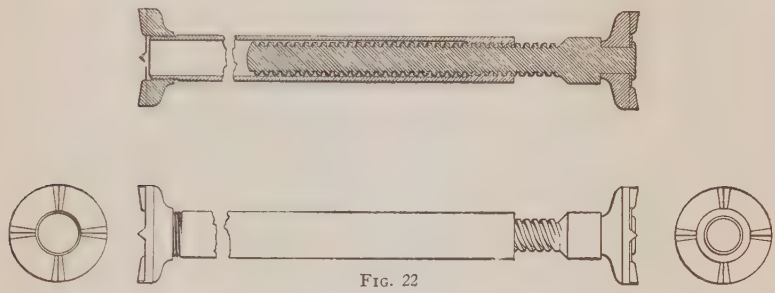

FIG. 22

two or three times, whereas the 1-inch material can be driven only once. A length of about 7 or 8 feet is generally to be preferred for vertical piling. When the trench is deeper than this, longer planks are used or two or more sets of sheeting are required, each row being driven inside of the next row above, as shown in Fig. 21, where planks *b* are inside of planks *a* and planks *c* are inside of planks *b*. The sheeting planks are sharpened at their lower ends so that they can be driven into the ground more readily. Usually, the excavation follows closely, or precedes slightly, the driving of the sheeting, and the sheeting has a chisel edge, the bevel side being turned toward the center of the trench. Where it is necessary to drive the sheeting far ahead of the excavation, as in very soft soil, there should be a double bevel so that the piling will sink vertically. At the beginning, the top of the sheeting is several feet above the ground, and it is usually necessary to construct a platform on which the workman driving the piles may stand.

The rangers *d* are generally about 4 in. $\times$ 6 in. in cross-section, and the cross-braces are either timber shores, as in Fig. 21, or iron screws of the type shown in Fig. 22 The timbers must be cut to the proper length and driven into place; they often

become loose, and must be wedged in or replaced by longer pieces; and they can generally be used only once. The iron screws, on the other hand, can be adjusted to fit any width of

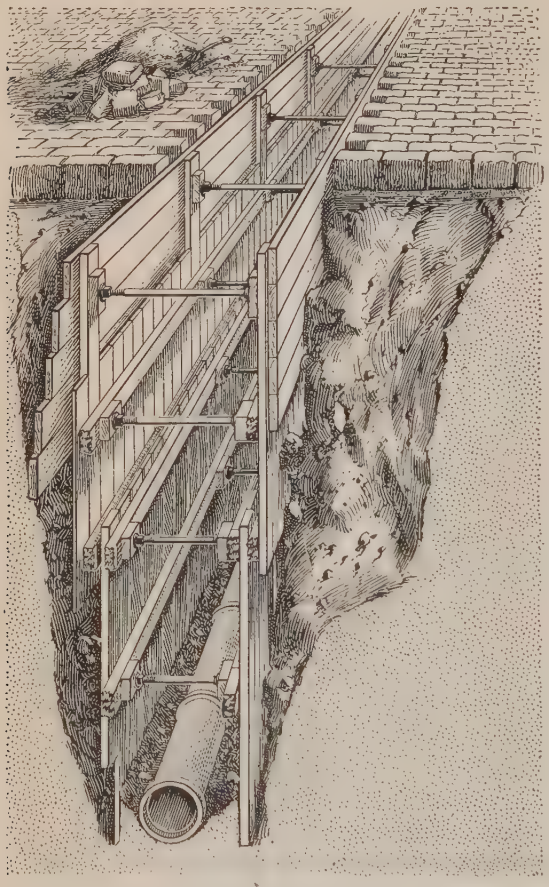

Fig. 23

trench within reasonable limits; can be put in place quickly, and tightened or removed without jarring the banks of the trench; and can be used any number of times. When timbers are used, they should be cleated, as at *f*, Fig. 21, so that they will not fall out.

Since the trench in Fig. 21 is very deep, staging platforms *g* are necessary. These are constructed on the bracing and extend half way across the trench. They can be used by the workmen in driving the lower sets of sheeting, as well as in removing the excavated material.

61. Box Sheeting.—In some cases, it is impracticable to construct a platform alongside the trench to permit the driving of vertical piles at the beginning. Under such conditions, it is best to place the planks that retain the banks in position horizontally for about 4 feet below the ground, as shown in Fig. 23, and to drive vertical piling for the lower portion. The construction with horizontal planks is called box sheeting. These planks may be in the ordinary marketable lengths, 16 feet usually being convenient. They should be 2 inches thick.

62. Stay Bracing and Skeleton Sheeting.—Where the banks of a trench require some support, but not enough to justify the construction of complete sheeting, stay bracing or skeleton sheeting may be used. *Stay bracing* consists of a pair of vertical planks, placed opposite each other, one against each side of the trench, and held in position by cross-braces; they are spaced from 6 to 12 feet apart. Stay bracing is for temporary use only, and, except where conditions are extremely favorable, should not be employed in a trench that will be kept open for more than a day or two.

Skeleton sheeting, as the name implies, consists of the usual rangers and braces, with vertical planks at intervals of 3 or 4 feet, or even at every brace. If needed later on, intermediate planks may be driven behind the rangers, and complete sheeting thus formed.

FOUNDATIONS

63. Foundations for Pipe Sewers.—Although circular sewers are not well suited for use in soft ground, sizes of pipe less than 12 inches in diameter do not usually require a foundation other than the natural earth at the bottom of the trench to prevent failure. In mud, however, even small sewers require an artificial foundation to prevent uneven

settlement. The simplest method of distributing the load is to level the bottom of the trench and to lay 1 in. × 12 in. boards end to end, the splices being made by boards underneath. Since wood rots quickly when alternately wet and dry, boards should not be used unless they will be permanently under water. When boards are not suitable, a concrete foundation must be used, as shown in Fig. 24.

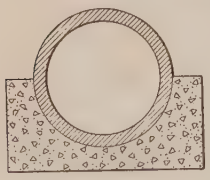

Fig. 24

Sometimes, the ground is sufficiently compact to support the pipe, but becomes so soft during the excavation that the bottom grading and the position of the pipe are uncertain. In this case, 4 or 5 inches of mud should be removed and gravel substituted.

64. When a large pipe is placed in a deep cut or in soft or treacherous soil, the trench bottom requires careful preparation. Just when an artificial foundation is necessary cannot be made a matter of formula or theory. Aside from experience, the only guide is to make the following test: Stand a piece of timber a foot square on end in the trench and load it with two or three times the weight that a foot of pipe is expected to carry. If there is continuous settlement, some foundation should be provided. The best method is to bed the pipe in concrete as shown in Fig. 24. All mud should be removed from the trench bottom before the concrete is deposited, as otherwise it will be forced up into the concrete. A fresh layer of concrete should be put in the trench just before the pipe is laid in place. If the ground is very wet and unstable, the pipe may be supported on timber piles in the manner shown in Fig. 25.

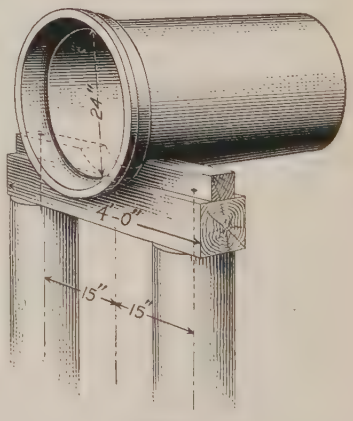

Fig. 25

65. Timber Platforms.—If the ground at the bottom of a trench is very wet and unstable, a wooden platform may have to be built to support even concrete. Longitudinal timbers about 4 in.×8 in. are worked down into the earth, and a platform is built on them, much like a wooden sidewalk. If more than this is needed, the longitudinal timbers are supported on piles. In Fig. 26 is shown the construction for a circular sewer with a concrete invert and a brick arch; and in Fig. 27, the construction for an egg-shaped sewer is shown. Standard practice is illustrated in both cases.

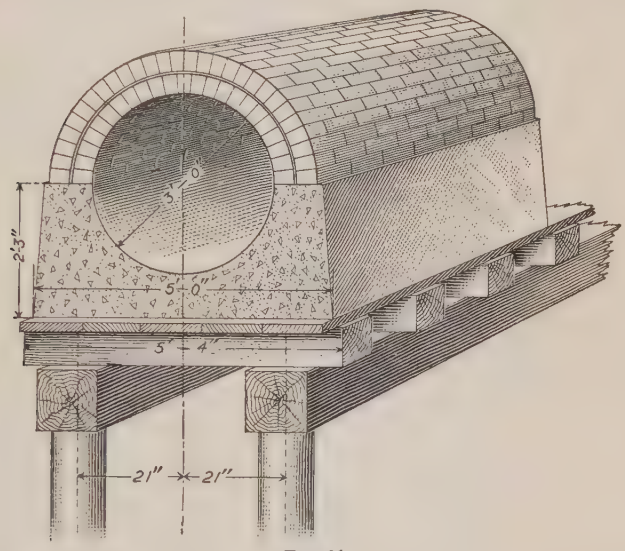

FIG. 26

66. Cradles for Brick Sewers.—Ordinarily, a brick sewer is built directly on the natural soil, the space outside of the sewer being filled with earth as the courses of brick rise from the invert on each side. This is not, however, a very firm construction, since the backfill cannot be tamped without disturbing the green masonry, and is very liable to settle and allow distortion of the cross-section of the sewer. Therefore, especially if the ground is soft, a *cradle* is often used. This is a wooden or concrete structure, so built as to rest securely in the bottom of the trench and to carry the brick on its interior

surface. A wooden cradle is generally made up of lagging supported on a series of **U** frames as shown in Fig. 28. The inside of the cradle should have the same diameter as the out-

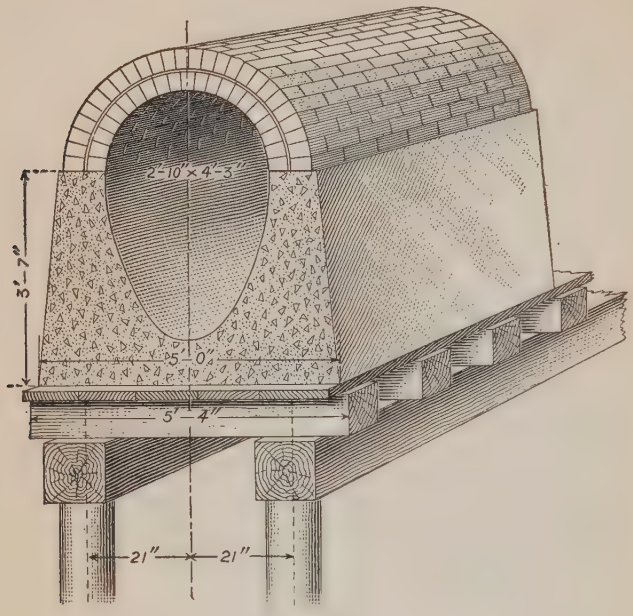

Fig. 27

side of the sewer, so that the bricks can be laid directly against the wood. The **U** frames are spaced about 24 inches apart, and the cradle sections are usually 8 feet long. They are constructed on the ground near the work, lowered to place in the trench, and blocked up to grade. Then, the earth is tamped solidly behind the cradle, and the bricks are laid on it.

Fig. 28

A wooden cradle should not be used in a dry trench nor in a trench that is alternately wet and dry, because under these conditions the wood will decay and allow the sewer to settle. Concrete

cradles are used instead of wood in such situations. Concrete is also better for a circular sewer in poor ground where uniform distribution of the load on the foundation is necessary to prevent uneven settlement.

MISCELLANEOUS OPERATIONS

67. Removal of Water from Trench.—Water is one of the troublesome factors in trenching, and its removal from the trench is necessary for good work. If water occurs in small quantities, a tin boat-pump may be used; much better is a hand-operated diaphragm pump which will handle 30 to 50 gallons of muddy water per minute. Larger sizes of diaphragm pumps are operated by gasoline engines. Their most effective depth is less than 15 or 20 feet. Where steam is available, centrifugal or vacuum pumps are used. A small and portable gasoline-driven centrifugal pump, which may be lowered into a trench or sump, is often suitable.

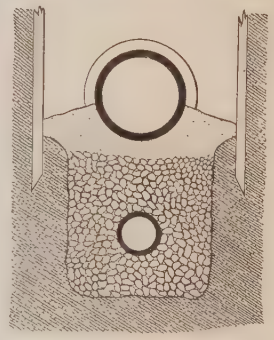

FIG. 29

The most effective method of drainage is to install a system of drain tile below the line of the sewer, either directly beneath, or to one side. These tiles may be agricultural drain tiles, or bell-and-spigot pipe, 4 inches in diameter for most purposes and 6 inches for the lower ends of long lines. These drain lines are laid with open joints and are surrounded with gravel to keep out dirt. They discharge into a natural watercourse, if the topography allows, or else into a drainage sump, from which the water is pumped. Where the ground water is high, the drains should be made permanent, with a natural outlet if possible. The construction of a drain is shown in Fig. 29.

68. Advantage of Drains.—Often, where pumping is depended on for removing water, the water collects in the trench, saturates the banks during the night, and seeps out

during the early part of the day, interfering with the work. It also keeps the bottom of the trench soft, and, under the treading of the workmen, mud is formed, which is not suitable as a foundation for the pipe. Where tile drains are laid, the trench will be kept free from water throughout the entire 24 hours, the banks will gradually drain, and the bottom of the trench will be kept in a much better condition.

69. Back Filling.—Earth, from which large lumps and stones have been excluded, should be packed and tamped carefully around the sewer in thin layers, the filling being kept to the same level on both sides. After the earth has been well packed in this way to a height about 1 foot above the sewer, the remainder of the back filling may be done with more or less care, according to circumstances. In paved or important and much-traveled streets, the earth should be compacted all the way to the surface. Ordinarily, the back-filling is rammed in layers; but in sandy or gravelly soils, it is preferable to flood the trench with water, as this settles the material more effectually than ramming. In less important streets, particularly when the soil is sandy or gravelly, less care in compacting the upper part of the filling is necessary, and the earth may be rounded over the refilled trench and allowed to settle by itself. When the sewer is not in a street, the back filling may be done with a horse-drawn scraper. In back filling, care should be taken not to dump heavy objects over or on the sewer in such a way as to injure it.

SEWERAGE
(PART 3)

SEWER APPURTENANCES

MANHOLES

1. Introduction.—In order that a sewerage system may function with the least amount of trouble, various appurtenances are needed. These include *manholes,* which are shafts or chambers allowing access to the sewers; *lampholes,* which allow inspection of the sewer from adjacent manholes by means of lights; *inlets,* which allow the entrance of storm or surface water to the sewer; *catch-basins,* which catch and retain some of the solid material carried by storm water, thus lessening the danger of clogging up the sewer; *flush tanks,* by means of which a large flow of water is discharged at intervals to wash out deposits in the sewer; and *overflows* and *regulators,* which control the flow of sewage.

2. Use and Location of Manholes.—A manhole, leading from the surface of the ground to a sewer, is used for purposes of inspection and cleaning. An inspector can get down into a manhole, and, by looking through the sewer pipe to a light at the next manhole, tell whether the pipe is obstructed or not. If it is, the obstruction may be removed by running scrapers or brushes through it.

In order that workmen may use manholes for cleaning purposes, the distance between manholes must not exceed certain limits. For small pipe sewers, the manholes may be placed

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from 300 to 400 feet apart, as that is about the greatest distance through which a jointed stick can be forced. On sizes 24 inches or larger the manholes may be 600 feet apart. Their position is also determined by the requirement that there shall be a manhole at every junction of two or more sewers, at every change of grade of every sewer, and at every change of direction. These requirements make it necessary that the sewer shall run in a straight line from manhole to manhole, and that the changes in line and grade shall come at or in the manholes. Since sewer crossings usually occur at street intersections, man-

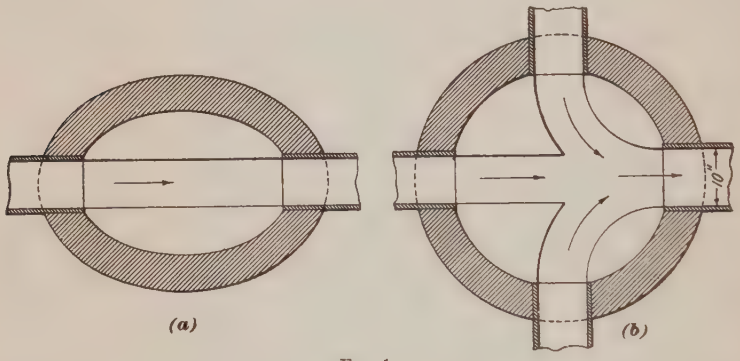

FIG. 1

holes are generally placed at such intersections, and, in long blocks, also midway between. When the street grade changes within the block, a manhole is usually added, no matter what the length of the block may be.

3. Shape of Manholes.—Manholes are made either bottle-shaped, or with the walls vertical, or nearly so, and the uppermost 2 or 3 feet domed or stepped in. The inside diameter at the top is almost always 24 inches, and the diameter of the bottom depends on the size of the sewer. For small sewers on a straight line the bottom may be either a circle about 4 feet in diameter, or an oval 4 ft.×3 ft., as shown in Fig. 1 (*a*). Where an intersection occurs, the bottom is circular, as shown in (*b*). The latter illustration represents a 4-foot manhole on a small sewer at a point where two laterals enter the main line, and shows how changes of direction are made in the manhole.

The vertical section of the manhole may have any one of the shapes shown in Fig. 2. In (*a*) the bricks are laid horizontally,

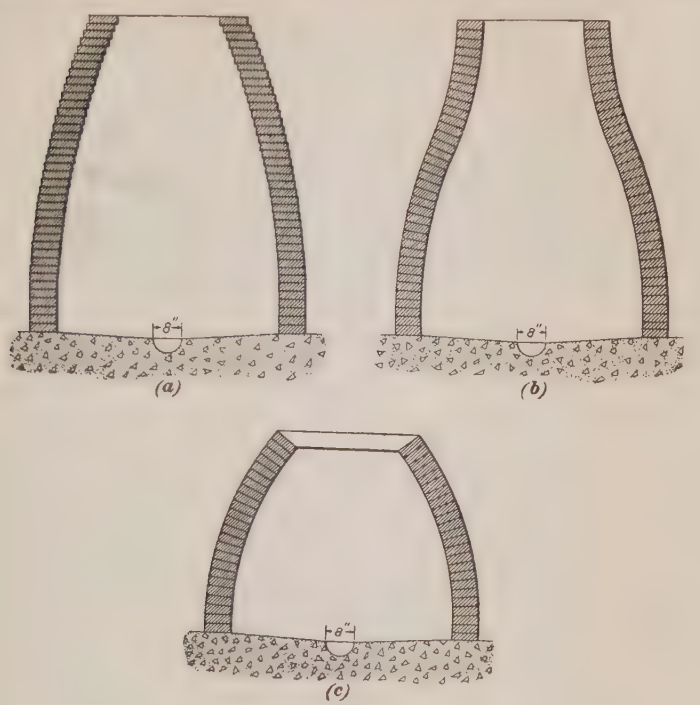

FIG. 2

the reduction in size at the top being made by offsetting each course just enough to obtain the desired reduction in diameter. In (*b*) the bricks are kept normal to the inside walls. In (*c*),

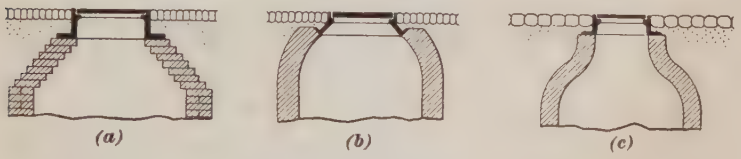

FIG. 3

which illustrates a form used only for manholes 4 feet deep or less, the courses are kept normal, as in (*b*), but are not brought back to horizontal positions at the top. The walls may also be

brought up straight and the bricks at the top laid according to one of the methods shown in Fig. 3, which is taken from *Sewerage and Sewage Disposal,* by Metcalf and Eddy. The type of manhole shown in Fig. 2 (*a*) requires the most bricks

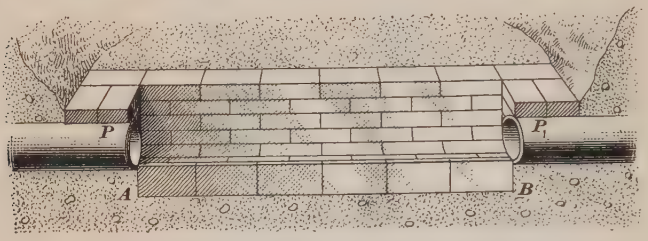

FIG. 4

and has the most room at the bottom; it is generally used for depths over 5 feet.

4. Construction of Bottom of Manhole.—The bottom of a manhole may be made either of brick or of concrete, the latter being the simpler construction. Fig. 4 illustrates how a brick bottom is started. The ends P and P_1 of the pipes are left the proper distance apart and protrude from the bank of dirt that is thrown back into the trench. A row of bricks on edge is first

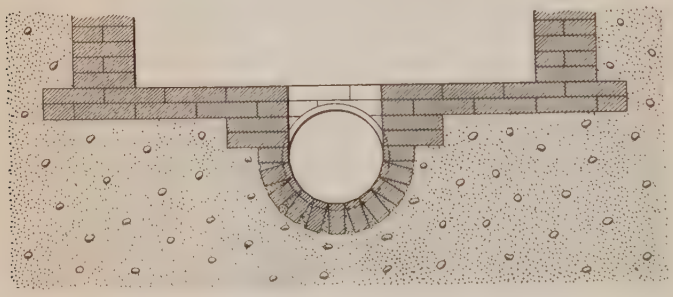

FIG. 5

laid from A to B, the tops forming a continuous line from the invert of the pipe P to the invert of the pipe P_1. Where a change of grade occurs, this line of bricks becomes a vertical curve. Against this line, another row of bricks is laid, being held up on the back by dirt tamped in and by a layer of mortar.

These rows are carried up on each side to the horizontal diameter, conforming to the curvature of the pipe. Beyond that height they rise vertically to the top of the pipe, as shown in section in Fig. 5. The inside wall of the manhole is brought just flush with the ends of the pipes, and a brick floor is carried around on the refilled earth.

When concrete is used, a mass is dumped down in the bottom of the manhole excavation; it is leveled up for a floor at about the level of the top of the entering pipes, and a channel is formed in the concrete by a trowel and by hand. A wooden form may be used, against which concrete is packed, but a good workman using damp concrete can shape it without the form. Sometimes a split cast-iron pipe, that is, a pipe split longitudinally (easily done with a cold chisel) is put in through the manhole, and the concrete packed against it. The concrete should extend outwards to support the manhole walls and project about 6 inches beyond them. Care should be taken not to have a greater load per unit area on the manhole than on the sewer, or uneven settling may break the connection between the manhole and the sewer.

5. Materials Used for Manhole Walls.—The walls of most manholes are made of brick, though concrete is used to some extent. The cost of constructing forms for concrete manholes, perhaps all of different depths and shapes, may be considerable, and, therefore, it is generally cheaper to use brick. Steps should be provided in all manholes, and these are placed easily in brick manholes by bedding iron inserts in the mortar joints.

6. Thickness of Walls.—No satisfactory theory is available for determining the thickness of manhole walls. Under ordinary conditions, they are two bricks, or 8 inches, thick. For shallow manholes built in dry clay, a 4-inch wall may be used, but it is not a safe construction. Where a high head of ground water exists, where heavy lateral pressure is expected, and where the manhole extends more than 15 feet below the surface, the walls should be made 12 inches thick.

7. Manhole Frames and Covers.—The walls of a manhole are capped with a cast-iron frame and cover, the latter being

flush with, and a part of, the street pavement. The frames are usually 8 to 10 inches high to allow paving to be laid on top of the brickwork and flush with the cover. Where extra-heavy paving is needed, as in business and trucking districts, the cover should be as much higher as may be necessary to allow the paving to be laid. In Fig. 6 is shown a frame *F F* and a cover *C C*. The base *B B* of the frame is from 4 to 5 inches wide and

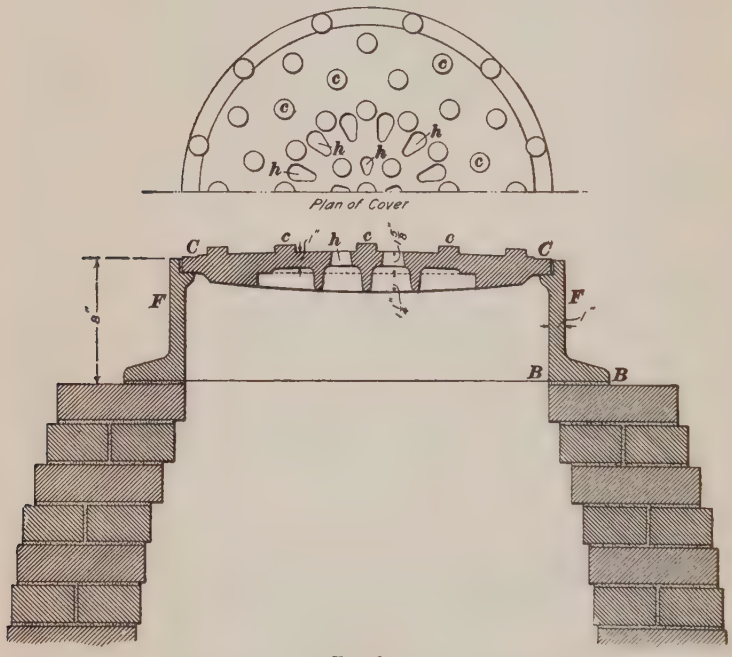

Fig. 6

rests on a bed of mortar directly on the brickwork. The thickness of the cast-iron frame is generally 1 inch. The holes *h* in the cover are used for ventilation, and the projections *c* prevent the top from wearing smooth and becoming slippery. The frame weighs about 200 pounds and the cover about 150 pounds. They may be designed by the engineer and cast in a local foundry or may be bought in standard designs from large foundries.

For the manhole shown in Fig. 2 (*c*), a special frame must be designed to fit the inclined brickwork and have the top hori-

zontal. The cost of such a casting is large, and it is better to avoid this form of construction.

8. Special Sizes of Manholes.—At intersections on 18-inch to 36-inch sewers, the bases of manholes may be 6 or even 8 feet in diameter, in order to bring the curves of the sewers inside the manhole. To find the diameter required, it is best to make a drawing of the lines of pipe meeting at the manhole, sketch in the curves necessary to make the flow lines meet smoothly, and make the diameter of the manhole bottom sufficiently large to include all the curves. In Fig. 7 is shown a junction of two 24-inch sewers and one 18-inch sewer with a

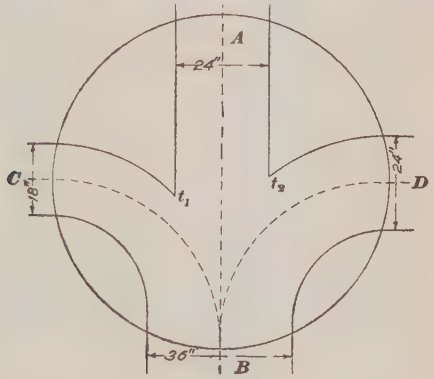

FIG. 7

36-inch sewer. The curves must not be too sharp, and the tongues t_1 and t_2 must project into the main sewer far enough to guide the flow into the 36-inch main line without eddying or other disturbances. The large circle represents the inside of the manhole wall and is $7\frac{1}{2}$ feet in diameter. As will be explained further on, it is sometimes necessary to have manholes of very large size, which will contain gates or valves, or will allow the making of connections between sewers at different grades.

9. Connections Between Sewers at Different Grades. Where the connecting sewers, as shown in Fig. 7, are at about the same level, the connections are easily made; but it frequently

happens that, on account of topography, the main sewer $A\,B$ is deep at its upper end, whereas the branches C and D, perhaps extending across a valley, need be only a few feet deep. The problem is to connect the flow of C and D smoothly with the flow of $A\,B$. In Fig. 8 is shown, to a smaller scale than is used

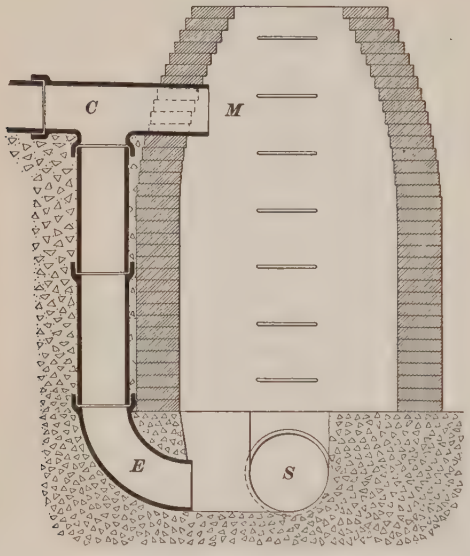

FIG. 8

in Fig. 7, the usual construction for these conditions. The pipe line C ends in a T or Y so placed that the branch comes just outside the manhole wall. A vertical pipe then carries the flow down to the main sewer below. The opening M serves for inspection and cleaning. The T pipe C must be carefully supported and the elbow at the bottom should be buried in concrete to carry the weight of the vertical pipe. From the lower end of the elbow E to the main sewer S, a channel is made in the concrete floor of the manhole.

10. Manholes for Large Sewers.—Sewers 3 feet or more in diameter are large enough to be entered, and curves of long radius may be used. Manholes are necessary only to provide access to the interior and will generally rest with their walls

directly on the arch of the sewer. In this case, the sewer must be designed to bear the load of the manhole. For sewers up to about 6 feet in diameter, the manhole should be symmetrical with respect to the center line of the sewer, as shown in Fig. 9 (*a*); for large diameters, the manhole may be built with one wall tangent to the side of the sewer, as shown in (*b*).

11. Lamp Holes and Fresh-Air Inlets.—Where it is necessary to make a change in the grade or alinement of a pipe sewer between two manholes a short distance apart, it is economical, and sometimes satisfactory, to use a lamp hole instead of another manhole. Lamp holes are also used, but more rarely, where the sewer is straight for a considerable distance beyond the usual spacing of manholes. A lamp hole is formed by placing a

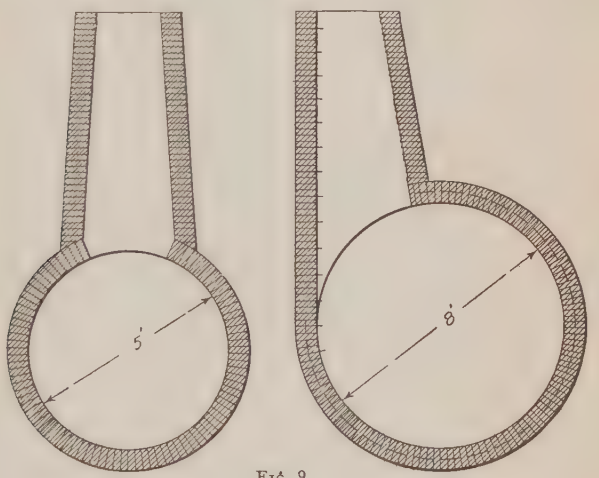

FIG. 9

T in the sewer pipe and inserting a vertical pipe in it, as shown in Fig. 10. The sewer may be examined from the adjacent manholes to determine whether an electric light lowered through the vertical pipe is visible.

In paved streets the vertical pipe should extend nearly to the surface and should be covered and protected with a heavy cast-iron frame and cover, as shown in Fig. 10. In unpaved streets, the top of the pipe should be left about a foot below the surface

and it should be covered with a light casting or with a terra-cotta cap. In all cases the T should be well supported and buried in a mass of concrete to prevent the weight of the vertical pipe from breaking it. The concrete should, therefore, be brought well up on the pipe, as shown in Fig. 10. The iron casting should be supported separately on a bed of concrete, so that the pipe will not settle with it.

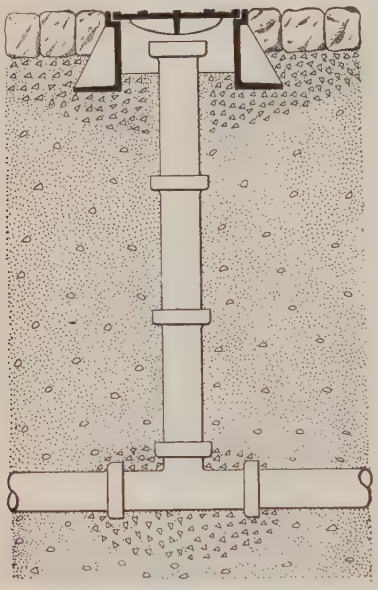

FIG. 10

If the cover of a lamp hole is perforated, it will assist in the ventilation of the sewer. The pipe is then called a *fresh-air inlet*. Lamp holes and fresh-air inlets can be constructed cheaply; but since the sewer cannot be cleaned from them, they are not used much in present-day practice.

INLETS AND CATCH BASINS

12. Inlets.—Inlets are provided at frequent intervals to allow surface water from the gutters to reach the sewers and thereby keep the gutter and the street free from such amounts of water as would hinder either pedestrians or vehicular traffic. They are generally located at all low places and at such other places as may require them. The distance between them is usually made less than 300 or 400 feet. They may be placed in the curb or in the gutter. If in the gutter, grates or bars capable of sustaining heavy loads must be used, and the openings

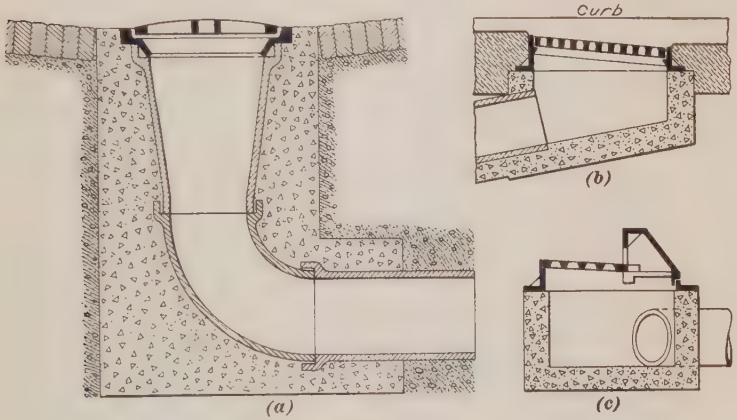

FIG. 11

cannot be large; if in the curb, they need not be designed to bear such a load, but bars or guards are usually provided. These may be spaced very closely to prevent the entrance of such trash as sticks, twigs, etc., or just closely enough to prevent children from falling or crawling into the sewer.

In some cities, inlets are located at the angle of street intersections, but in such places they are subject to heavy wear from trucks cutting the corner too closely. An inlet at each side of the corner just before the cross-walk is reached is preferable.

In Fig. 11 (*a*) is shown an inlet used by Metcalf and Eddy for parks and light-traffic drives; and in (*b*) and (*c*) are shown curb inlets used in Bronx Borough, New York City.

13. Catch Basins.—The purpose of a catch basin is to admit storm water to a sewer without allowing too much debris from the street to enter, and without permitting sewer gas to escape. Therefore, catch basins are used only in combined sewerage systems. They were formerly considered an indispensable part of such a system, but in modern practice their

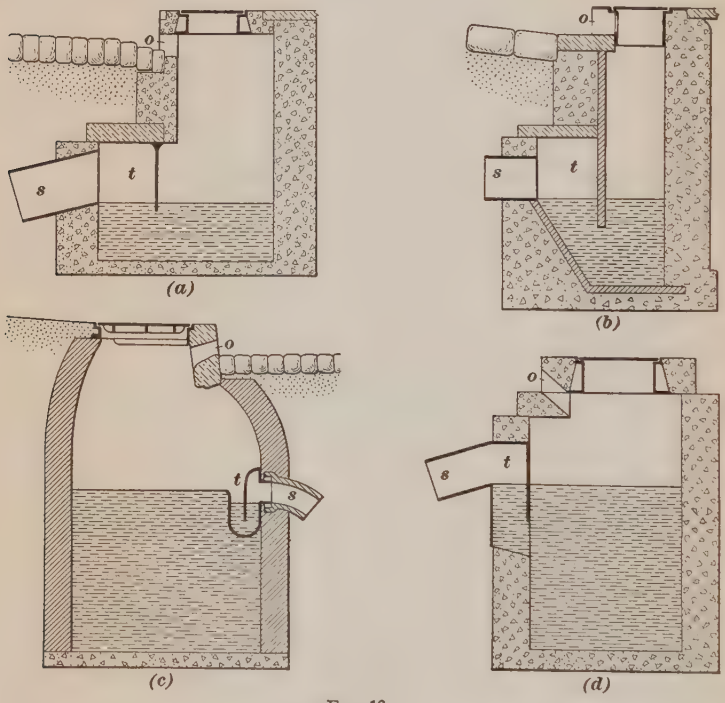

FIG. 12

value is questionable. Reasons for the decrease in their use are: better paved streets with less grit and debris to be carried into the sewers by storm water; sewers laid on grades to furnish a self-cleansing velocity; and the realization that few cities clean their catch basins often enough to enable them to function in the intended manner.

Catch basins, when used, are placed at street corners and, sometimes, if the block is long, at intermediate points also. The

forms of basins most common in the United States are shown in Fig. 12. Each consists of a chamber, or basin, into which the storm water flows from the street gutters through the opening *o*, and from which it is discharged into the sewer through the outlet *s*. The outlet is placed some distance above the bottom of the basin so that not all of the coarse and heavy matter suspended in the storm water will enter the sewer, but some will settle to the bottom of the basin and be retained there. In order to prevent sewer gas from entering the catch basin, and perhaps escaping in the vicinity of the sidewalk, where it is objectionable, the outlet to the sewer is designed so as to form a trap as shown at *t*, Fig. 12.

The total depth of a street catch basin should be from 6 to 8 feet and, to avoid freezing in cold climates, the water level should be from 2 to 3 feet below the street surface. It is important that catch basins should be water-tight to keep the water level high enough both to seal the trap against the escape of sewer gas and to cover the deposit of mud so that it will not give off disagreeable gases. Even in a water-tight basin, decomposition of organic matter may take place and odors thus be caused; or, because of evaporation, the water level may sink below the proper level. Catch basins are generally built of brick or concrete. When built of brick, they should be lined with cement and plastered both inside and outside with cement mortar so that they will not leak. The construction may be varied in detail to suit local conditions.

FLUSH TANKS

14. Object of Flush Tanks.—Flush tanks are used either to collect the sewage and discharge it rapidly at intervals for the purpose of flushing the sewers or, more often, to collect and discharge water from some public supply for the same purpose. Flushing is required only at dead ends and at the upper lines of sewers, particularly where the grades in these parts of the system are slight. Farther down the line the accumulated flow should be sufficient to keep the sewers clean if they are laid on proper grades.

Where flushing is necessary, the sewers should be flushed regularly, and, therefore, automatic appliances are most convenient and are generally used for this purpose. Sometimes flush tanks are omitted on steep grades, and the sewers are flushed out occasionally by applying water from a hose through lamp holes placed at dead ends.

15. Operation of Flush Tanks.—The requisites of an automatic flush tank are certainty of action, rapidity of discharge, ease of inspection, durability, and economy in first cost and maintenance. Siphon tanks, which discharge by means of

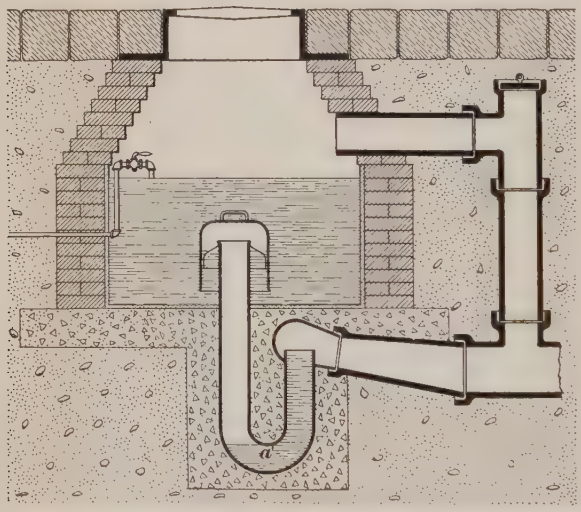

FIG. 13

siphons when they become filled to the desired point, are used almost exclusively, as moving mechanical parts are objectionable in a flush tank. The type of siphon shown in Fig. 13 consists of two simple castings; namely, a U tube, or trap, with a mouthpiece, and a cast-iron bell placed over the longer leg of the siphon and held in place by brackets cast on the trap.

The action of the siphon is as follows: As the water entering the tank rises above the lower edge of the bell, it compresses the air within the bell. The lower portion of the trap contains

water, and as the water level in the tank rises, the confined air gradually forces the water out of the long leg of the trap. As long as the water level in the long leg is above the point *a*, the difference between the water levels in the two legs equals the difference between the level of the water in the tank and the level of the water within the bell. The column of water in the short discharge leg above the level of the water in the long leg, therefore, counterbalances the column in the tank above the level within the bell. However, as soon as the water level falls below *a*, a portion of the air rushes around the lower bend and carries upwards with it some of the water in the short leg. The equilibrium is thus destroyed, and the contents of the tank are discharged.

16. Hand Flushing.—In some places what is called hand flushing is practiced. This is generally done by attaching a hose to the nearest fire hydrant, and inserting the hose into a manhole. Sometimes a small pipe, on which a valve or stop is placed, is run from a nearby water main to the manhole. When the valve or stop is opened, water runs into the manhole, but the pipe does not deliver enough water to accomplish the work of flushing; a cover, stop, or valve must, therefore, be placed in the manhole over the end of the sewer, so that a reserve of water is built up in the manhole. When the cover or valve is withdrawn, the water rushes into the sewer. Such an arrangement is usually called a flushing manhole.

OVERFLOWS AND REGULATORS

17. Overflows.—When a large volume of storm water is flowing through a combined sewer, it is not desirable to treat or pump all of the sewage; so overflows are used in combined sewers to divert most of the storm-water flow from the sewer leading to the pumps or treatment plant, and allow it to discharge directly into a watercourse. The storm sewage diverted and discharged directly contains some house sewage, but it is assumed that so much dilution takes place that there is little danger from this house sewage. If an overflow is placed in an

intercepting sewer, the outfall line to the treatment plant need be designed to carry only the maximum flow of dry-weather sewage, whereas the storm water is diverted directly into the river or lake.

18. Diverting Weir.—The simplest method of handling storm-water flow is by means of a weir, called a *diverting weir,* whose crest is about at the level of the maximum house-sewage flow. When the flow becomes heavy from rainfall, the level of the sewage rises and the weir overflows. Some of the house sewage goes over the weir, mixed with the storm water; but under the conditions of probable dilution, there is little possi-

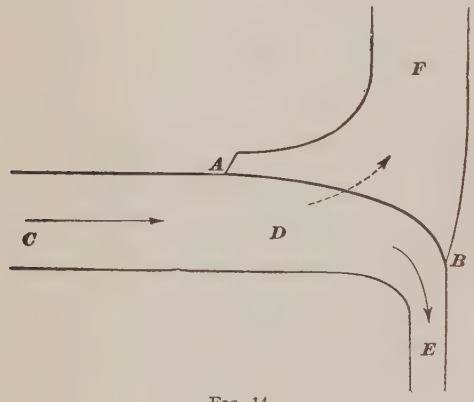

FIG. 14

bility of danger or nuisance from it. The length of the weir should be such that the depth of flow over it will not exceed a few inches.

In Fig. 14 is shown in outline the method of construction. *C D* is the combined sewer, carrying normally only house sewage, which flows into the small sewer *E*. In time of storm, the level of flow rises, and, after reaching the crest of the weir *A B*, most of the storm sewage runs off through the sewer *F* to the point of discharge, while only a portion of the flow continues through *E* to be pumped or treated.

19. Leaping Weir.—Another method of allowing the escape of the storm water is illustrated in Fig. 15. In dry

weather, the flow in the combined sewer $A\,B$ is small; the sewage falls directly into the sewer S at right angles to $A\,B$ and is taken away to the pumps or treatment works. In time of flood,

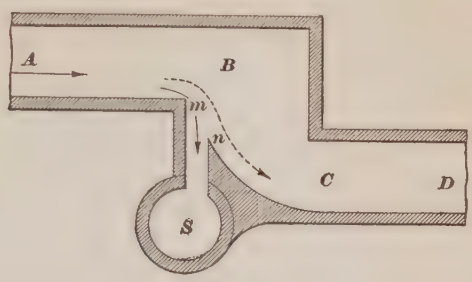

Fig. 15

the flow shoots across the opening $m\,n$, as shown by the dotted line, and goes off through the sewer $C\,D$ to the nearby point of discharge. This arrangement is called a *leaping weir*.

It is not practicable to compute exactly the width of the opening $m\,n$ in terms of the depth of flow in $A\,B$ and of the differ-

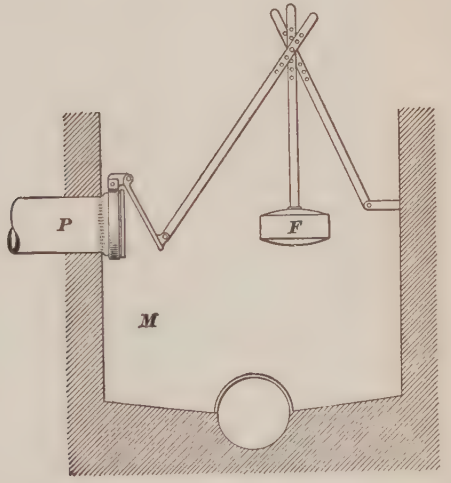

Fig. 16

ence in level between m and n; it is customary either to make the width of opening variable by providing a sliding crest at the point n, or to make the opening too small at first, and, after

observation of several storms, cut away as much of the masonry at *m* as is necessary. Rounding the crest at *m* has the effect of carrying more water through the opening. The edge of the opening at *m* is usually made horizontal, so that the cross-section of the sewer at that point has a rectangular invert.

20. Mechanical Diverter, or Regulator.—In a third class of overflow, known as a *mechanical diverter,* or *regulator,* a flap valve controls the flow of sewage to the direct-discharge line. In Fig. 16 is shown the simplest form of regulator. When the level of the sewage reaches the float *F* in the manhole *M* and continues to rise, the float is lifted and the valve at the end of the pipe *P* is opened. The position of the float is adjustable and can be regulated to start the discharge through the pipe *P* when the sewage level in the manhole reaches any desired height.

Regulators are used when it is desired to prevent damage to a sewer by surcharging or overloading. Whereas the overflow allows only the surplus to escape, the regulator may be so set that the entire storm flow is diverted to another outlet or discharged through special watercourses.

SPECIAL FEATURES

HOUSE BRANCHES

21. Size of House Branches.—Sewage from buildings is carried to the street sewers through drains called *house branches.* These house sewers are usually 4 inches in diameter, though they are occasionally larger. They should never be larger than the sewer into which they empty. It is well to adopt a uniform size throughout the entire system, or, if necessary, two or three sizes, as otherwise much confusion may occur later.

22. Grade of House Branches.—House drains require a greater rate of fall than the main sewers, because the flow in them is less constant and is smaller in volume. It has been learned by experience that the fall should not be less than 1 in

about 40 or 50, or about $\frac{1}{4}$ inch per foot. Where the inclination must be less than this, the drains should be laid with the greatest possible care and should be so arranged that stoppages can be removed with the least trouble.

23. Connections to Pipe Sewers.—House connections to pipe sewers are made by means of **Y** or **T** branches. A **Y**-branch connection is shown in Fig. 17; since the stub makes an angle of about 45 degrees with the sewer, a 45-degree curve or

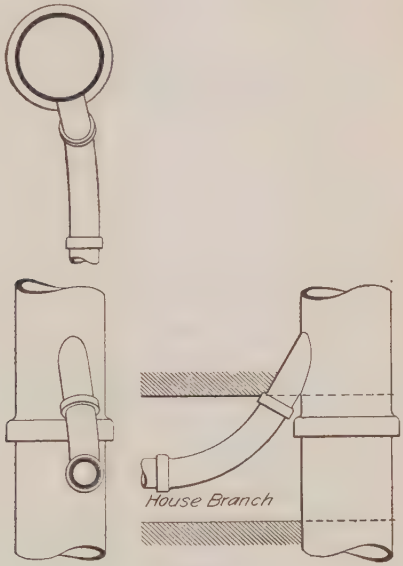

FIG. 17

bend, inserted in the stub opening, leads off about at right angles to the sewer. Generally, a **T** branch is preferred on a sewer larger than 24 inches in diameter and a **Y** branch on smaller sizes.

A connection should always be made by means of a branch, as a break in a pipe sewer for inserting a connection cannot be made water-tight. The branches should be inserted when the sewer is built, because if it is necessary to make a connection for which no branch has been provided, a length of sewer pipe must be removed and a branch put in its place. The drains to

the houses are laid after the sewer has been constructed; and, until the drain is put in, the end of the stub of the branch is closed by a cap or stopper, one form of which is shown in

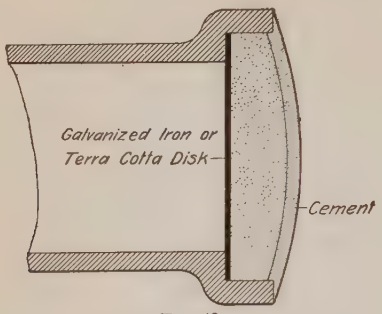

FIG. 18

Fig. 18. It consists of a metal or an earthenware disk outside of which is placed packed earth and a coating of cement. When the branch is to be opened, the cement is broken away and the plug removed.

When the grade of the house drain is not known, the inclination of the stub is based on judgment. If the trench is shallow and the house plumbing is a considerable distance from the sewer, the stub of the branch should be tipped up slightly; where the trench is deep and the building is close to the sewer, the stub is set at a steeper pitch.

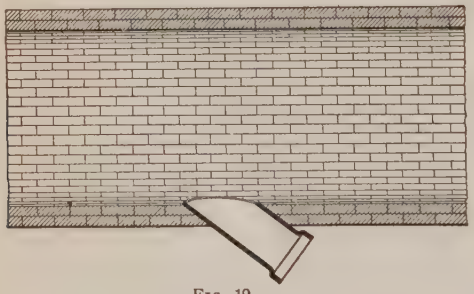

FIG. 19

24. Brick and Concrete Sewer Connections.—For making a house connection in a brick or concrete sewer, a piece of pipe, called a *branch* or *slant,* is used as shown in Fig. 19. It is

built into the side of the sewer just above the widest part. Where no connection has been provided in constructing the sewer, a hole may be cut through the sewer wall and a slant built firmly in place.

25. Laying House Branches.—It is best so to locate the branch in the sewer that the trench for the house drain will be approximately at right angles to the street. It is often necessary, when laying a house branch, to sheet up the sides of the trench; and the sheeting is much more easily placed and more secure when the trench is straight. The trench should be sufficiently wide next to the sewer to leave room for the connecting curve and to include the opening of the stub, as shown in Fig. 17.

SEWER INTERSECTIONS

26. General Requirements.—When two large sewers join, the sewer below the junction will ordinarily have a cross-sec-

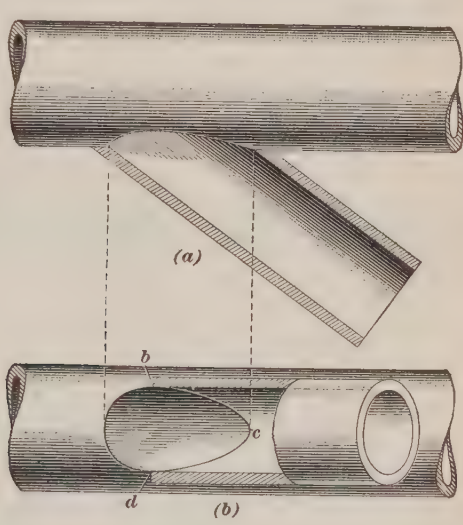

FIG. 20

tional area about equal to the sum of the areas of the two. In order to avoid a decrease in the velocity of flow of the sewage from either sewer, the two streams should meet with the same

velocity and at the same surface elevation. This is difficult, as conditions may change radically when different amounts of sewage are flowing. Other important features are to avoid weakening the sewers and to maintain ventilation in the upper parts of the sewers. The best circulation of air is secured by having the upper surfaces of the sewers continuous, but it is not always possible to obtain this condition. Careful design of a

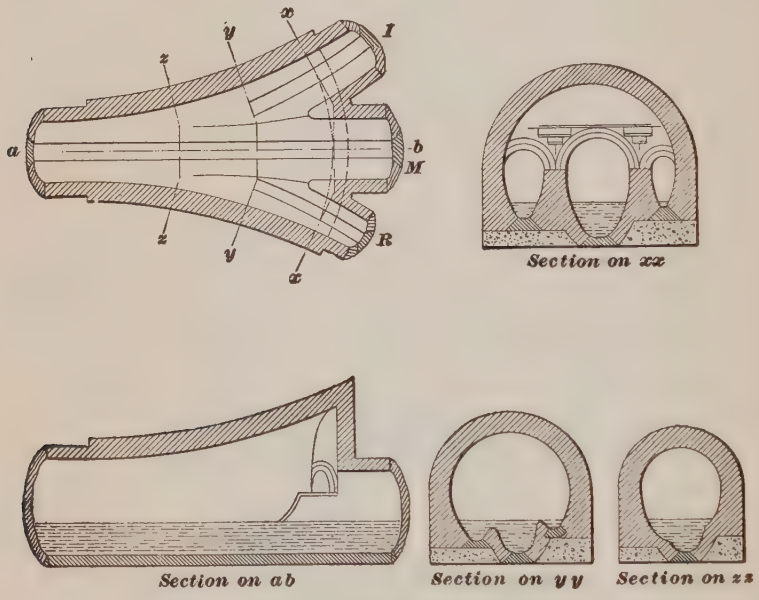

Fig. 21

junction is necessary to get a proper adjustment among conflicting requirements.

27. Construction at Junctions.—When the axes of two intersecting sewers make with each other an angle of more than 30 degrees, the intersection of the two cylinders is drawn out and a pattern made for its construction. In Fig. 20 (*a*) is shown the plan, and in (*b*) the elevation, of such an intersection, part of the branch sewer being cut away to expose the intersection *b c d*. This construction is objectionable where the sewers are large, because it weakens the sewers.

28. When the angle of intersection between two sewers is small, the junction is very long axially, and the sewer is weak over a great length. If the angle between two straight sewers is less than 30 degrees, or if a branch sewer is joined to a main sewer by a curve that is tangent to the main, some construction other than the plain intersection must be employed. In Fig. 21 is shown the *bell-mouth construction* which for many years was resorted to on such junctions. The main sewer M is joined by

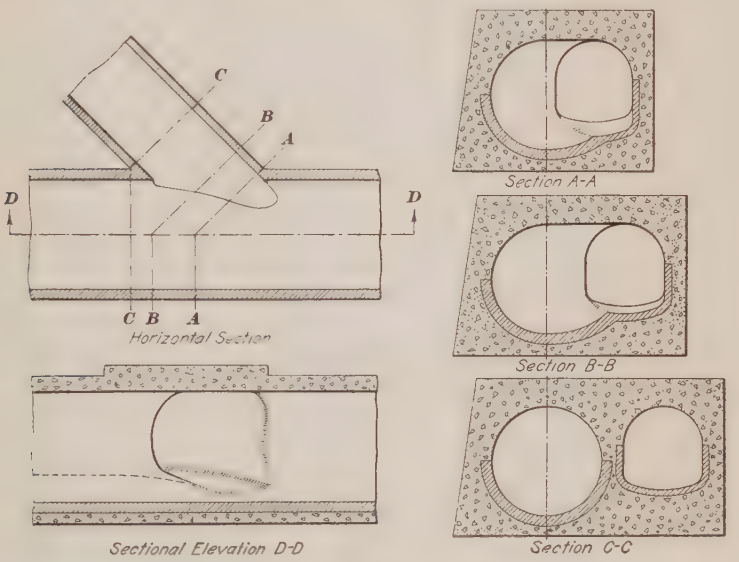

FIG. 22

two branch sewers L and R. The arches of all three sewers end just above the section taken at $x\,x$. At this point, an arch is turned from the outside of L to the outside of R, covering M entirely and rising a number of feet above the latter sewer. This large arch is then gradually reduced in size, being given the shape of a bell or trumpet, until at $z\,z$ it has been brought down to the same size as the arch of the main sewer below $z\,z$. The vertical section $a\,b$ shows how the height of the arch is reduced.

Owing to the skill required in constructing a bell mouth and to its cost, this connection has been largely superseded by the

flat-top junction. In place of the bell mouth, there is a flat roof between the tops of the arches, when they are at the same level, as in Fig. 22. When the tops of the arches are at different levels, as in Fig. 21, the roof is at the level of the top of the highest arch. The roof is generally constructed of concrete and is usually reinforced with steel beams to save headroom as well as to reduce the cost in construction.

STREAM CROSSINGS

29. Inverted Siphons.—A main sewer is often laid along the course of a small stream that follows, in an irregular way, a depression toward which all the smaller watercourses in the area tributary to the stream must flow. Such a stream generally crosses and recrosses the valley, flowing first along the foot of one bluff, and then along that of the other. It is frequently better, and sometimes necessary, to locate the main sewer along a more direct line than that of the stream, and this route requires that the sewer should cross the stream. It is also sometimes necessary that a branch sewer should cross a stream in order to discharge into a main sewer that lies on the opposite side of the stream. Occasionally these crossings may be made without depressing the grade of the sewer, but often the grade must be depressed, and what is called an *inverted siphon* constructed. In unfavorable ground, an inverted siphon is generally built of iron pipe; but if the ground is favorable and the work can be done readily in an open trench that can be kept free from water, the siphon may be built of concrete or of ordinary sewer pipe encased in concrete. The usual method is to build at each end of the submerged pipe a manhole, at the bottom of which the siphon pipe enters, the sewer itself entering higher up on the grade that is interrupted by the stream. In Fig. 23 is shown a design for such an inverted siphon.

30. Construction Features.—Siphons should be simple in construction, and the changes of direction should be easy and gradual rather than abrupt and sharp. Special attention should be given to eliminating possible obstructions to flow, and to

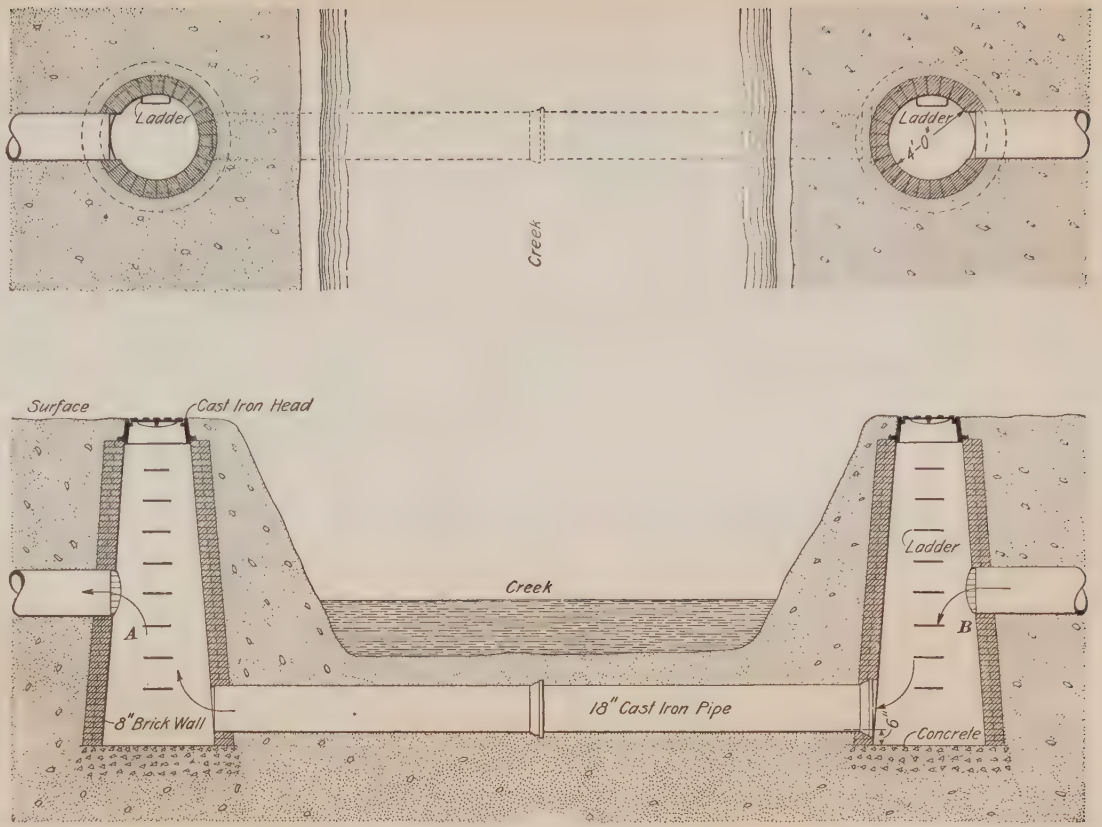
Ladder
Creek
Ladder
Surface
Cast Iron Head
Creek
Ladder
A
B
8" Brick Wall
18" Cast Iron Pipe
Concrete

FIG. 23

maintaining a uniform velocity and section throughout. A velocity of at least 3 feet per second should be maintained in the inverted siphon with average flow, because, when the velocity is less than this, deposits may form in the siphon. If the pipe is designed to carry the maximum flow, even at high velocity, the sewage at night may flow with such a small velocity that deposits will form. In order to avoid this difficulty, siphons are often built with two or three small pipes so arranged that they come into service singly, and in proportion to the amount of flow. Fig. 24 shows a very simple arrangement for this purpose. The circular pipe shown is the sewer entering the manhole; *A* and *B* are the two lines of pipe making the siphon. When but a moderate volume of sewage is running, the pipe *A* is the only one in action. When the flow increases so that the pipe *A* is surcharged, the level of the sewage in the manhole

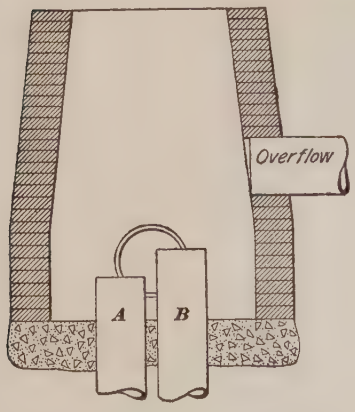

Fig. 24

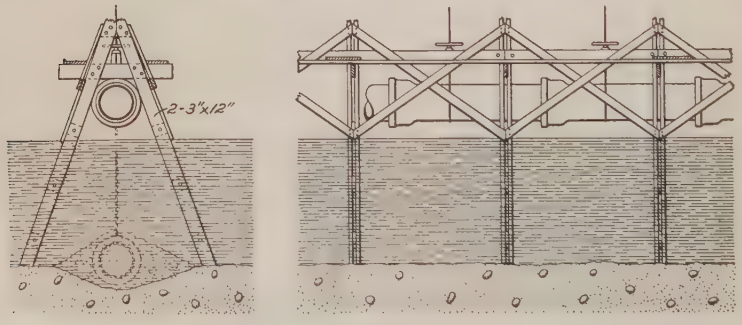

Fig. 25

rises to the top of the pipe *B*, and that pipe also comes into action.

The submerged pipe for an inverted siphon can be laid in the same way as a water main. Usually, trestle bents are built in

the stream, the pipe is suspended from them while being jointed, and the entire line is lowered into place without disturbing the joints. This type of construction is shown in Fig. 25.

31. Bridges for Sewers.—Across a deep narrow gorge it is often economical to build a light bridge on which the sewer pipe, carefully boxed to avoid freezing, is carried. In Fig. 26, taken from *Engineering Record*, is shown a bridge built for this pur-

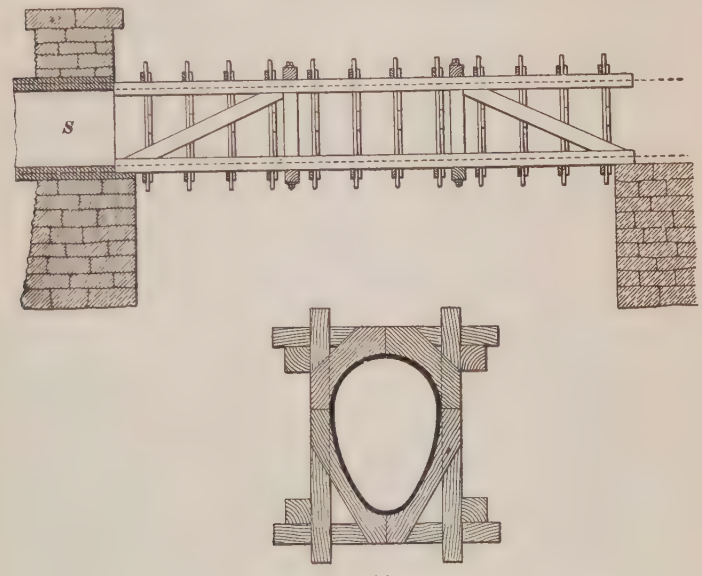

FIG. 26

pose. In this particular case, the sewer up to the bridge is of brick, 2 courses, or 8 inches, thick; on the bridge, the sewer is built of two thicknesses of tongue-and-groove yellow-pine flooring, with tar paper between. The light character of the structure should be noted. The truss is of simple construction and, except for two vertical steel rods, is composed entirely of timber. The sewer is held in place by boards sawed to fit the outside. On bridge crossings, however, cast-iron pipe is much more frequently used for small sewers and steel for large ones. Wrought-iron pipe has a short life and, for this reason, is seldom used.

It is not good practice to carry a sewer on the floor of a highway bridge. Lead joints in cast-iron pipes often leak after a short period of exposure to the vibration usually found in such a bridge. Moreover, to enable the sewer to pass over the bridge, it is generally necessary to introduce a rise in the grade which is considered objectionable by most designers because the solids in the sewage tend to remain behind

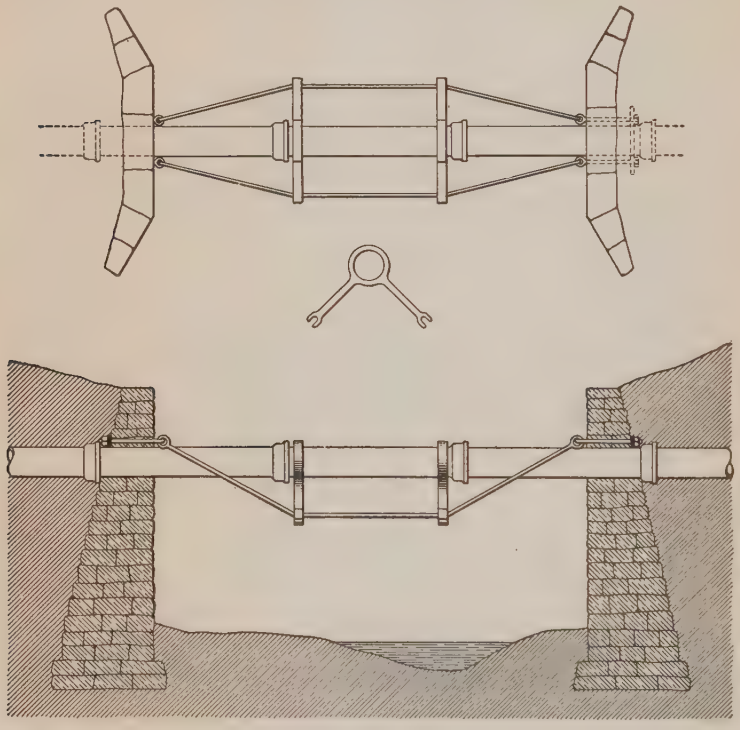

Fig. 27

In Fig. 27, which is taken from Folwell's *Sewerage*, is shown the method of carrying a sewer pipe on the piers of a highway bridge. By this construction, vibration, as well as the cost of a special bridge, is avoided. At each end of the middle length of pipe is a saddle-shaped straining piece, under which the bent tension bars of the truss pass; the ends of the bars are anchored in the abutments, as shown.

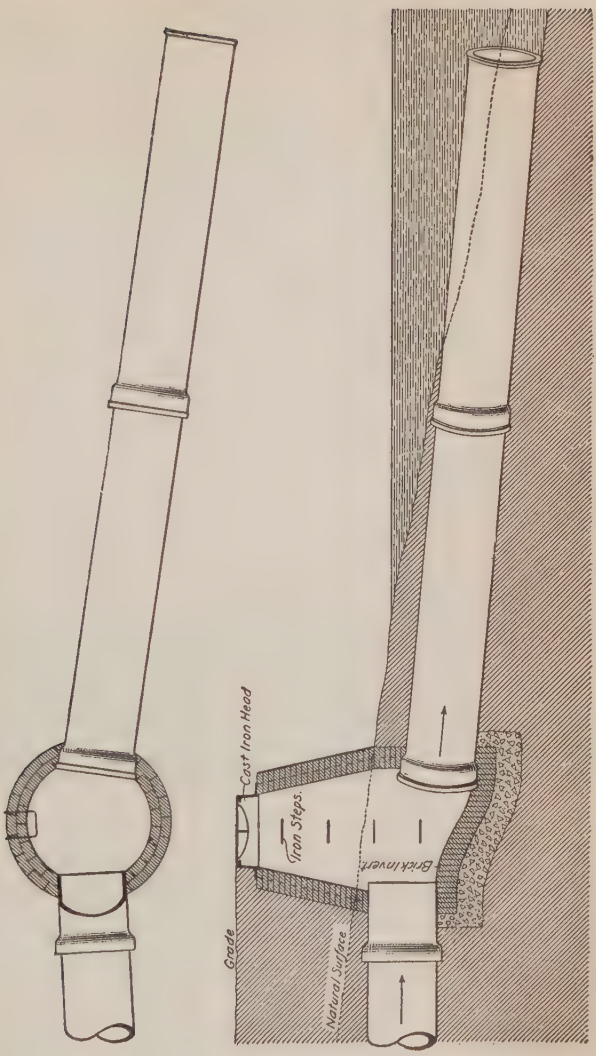
Cast Iron Head
Iron Steps.
Brick Invert
Grade
Natural Surface

Fig. 28

OUTLETS

32. Outlet for Separate System.—When house sewage is discharged into a large body of water, the outlet of the sewer should be made deep enough to be covered. If the sewer is in a separate system, a cast-iron pipe is carried out from a manhole on the bank. The pipe is buoyed up on rafts, jointed, and then sunk; or the joints may be cold-calked under water, as there is no need for water-tight joints; even pine wedges driven into each joint will answer the purpose. In a river with a strong current, or in a large lake, where winds may stir up heavy

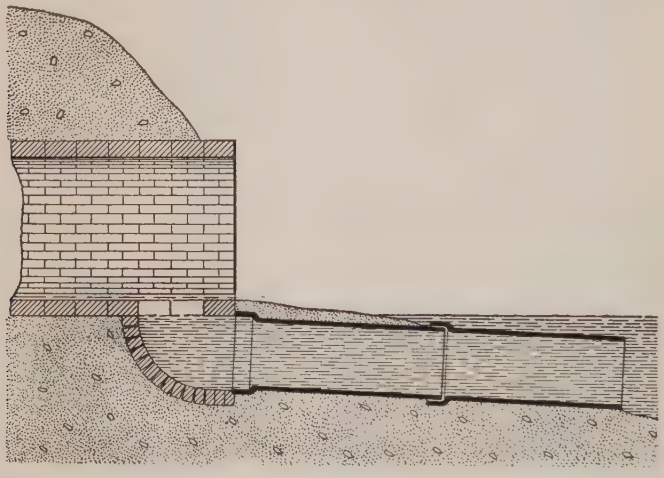

FIG. 29

waves, the pipe is fastened down by driving piles on each side of it at intervals of about 12 feet, and bolting timbers to the piles above and below the pipe. In a rock bottom, it will usually be sufficient to blast out a trench just deep enough to contain the pipe. In Fig. 28 is shown a simple design for an outlet.

33. Outlet for Combined System.—Where the sewer is in a combined system and the water near the shore is shallow, the great expense of carrying the large pipe out into deep water (the distance may be several hundred feet at time of low water)

may be avoided by carrying out a comparatively small pipe for the objectionable house sewage and letting the storm water discharge directly at the river bank. In Fig. 29 is shown the arrangement used for this purpose. The large sewer ends at the bank, with or without a masonry protection wall. A small iron pipe, of a size computed to carry the house sewage, leads out from the bottom and extends into deep water. In this way, the shallow water near the shore is not polluted with a continuous discharge of sewage during dry-weather flow, as it would be otherwise.

RECORDS AND ESTIMATES

SEWER RECORDS

34. Records of Sewer Lines.—The difficulty experienced in locating and constructing a sewer among works of which no record has been made, emphasizes the importance of keeping an accurate record of the exact location of every part of a sewerage system. This record should include the positions of the sewer lines with reference to the street lines; the grades of the sewers; the elevations at all changes of grade, junctions, and other important points; the exact locations of all catch basins, flush tanks, manholes, and lamp holes; and the positions of all Y branches or slants for house connections. It will also be well to keep a record of the positions of all gas, water, or other underground conduits encountered in the excavation.

35. Transit Notes.—Sufficient notes, clearly made and properly indexed, should be kept of all the field work to preserve a record of the locations of all important points, and of such other information as may be of value. In the notes, the starting point of the line at the outlet should be recorded as Sta. 0 (or Sta. 0+00).

The notes should give the position of the sewer line with reference to the street lines, and the position of the offset line with reference to the sewer line. The points where both lines of each street intersect the sewer line should be noted on the

latter line, and, in most cases, the offset distances to the street lines at such points should be given also. Offset distances to the street lines should be recorded for all points where angles occur in either the street lines or the sewer line. Measurements should refer to the sewer line, and not to the offset line.

36. Reference Points.—The sewer line should be so fixed by reference and measurements to permanent objects that its exact position may, at any subsequent time, be determined without difficulty. This is generally best accomplished by observing the points where the line of the sewer is intersected by the prolonged lines of the sides of buildings and by other well-defined lines of permanent objects, and measuring along the prolonged line the distance from the nearest corner of each object to the center line of the sewer. The buildings selected should be of a permanent character, such as brick or concrete buildings, and the measurements should be made to the nearest tenth or hundredth of a foot.

Another way is to have two pairs of reference points so located that the intersection of strings stretched between them will mark the center line of the sewer. Care should be taken to have these reference points so situated that, while convenient for use, they will not be destroyed by construction work.

37. Construction Notes.—The construction notes should contain a clear statement of what and where the work is; its character; the elevation of the surface and the sewer grade at intervals of 25 feet; the exact location of the sewer in the street; its size; and the exact location of all T and Y branches, slants, manholes, flush tanks, and similar accessories. Each of these appurtenances should be designated by a number to aid in keeping track of them.

If drain tiles are placed in the trenches parallel with the sewers, the notes should show their locations and sizes, and where they start and end. All branches that are built in manholes for future use should also be noted, with the sizes and elevations, and a sketch showing how they are placed.

If an artificial foundation is put in at any point, or sheeting is left in the trench, or any construction out of the ordinary is

used, the notes should show at what stations it is commenced and terminated; and, if a price has not been previously fixed for the work, and it is being done by contract, the notes should show the extra amount of material and labor and whatever else may be required for determining the compensation for the work.

The dates on which various parts of the work are done should be recorded. A note should be made of any unusual occurrence in connection with the work, such as an accident, refusal of the contractor to obey instructions, damage to the work by a storm, etc.

These construction notes will generally be transcribed on the final records by a person other than the one who took them in the field, and it is very important that they be complete and plain.

38. Location of Branches and Slants.—Y branches and slants for the convenience of property owners cannot be put in promiscuously, and often cannot be put in at regular distances. As the sewer is being laid, the engineer should examine the conditions of each lot, note the arrangement of the plumbing, select a convenient route for the house sewer, after consultation with the owner, and locate the slant or branch accordingly.

The position of each branch or slant should be temporarily marked with a stake just in advance of construction; and after the branch or slant has been placed, the exact station should be taken to the nearest tenth of a foot by dropping a plumb-line to the center of the opening.

39. Inspection.—It is necessary to see that the work is carried out properly and in accordance with the plans of the engineer, and that the specified amount and kind of materials are used. The duties of the inspector on sewer construction are detailed in Folwell's *Sewerage* as follows:

Be on hand before work is begun at morning or noon, to see that no partly set mortar is worked over, and that the new mortar is properly proportioned and mixed.

See that grade boards are not disturbed and that grade lines are taut.

See that no dirt enters or remains in the sewer while it is being laid.

Examine each pipe before it is lowered into the trench, and each load of sand, brick, cement, or other material as it is delivered on the work to see that the specified requirements are met.

See that each pipe or templet is set to grade and line, using a grade rod and a plumb-bob for this purpose.

Mark the position of each branch for a house or inlet connection by a stake driven in the bank directly opposite it, indicating the side of the street toward which the branch points. See that covers are cemented in each branch to prevent dirt or ground water from entering the sewer.

See that arch centers and concrete forms are set solidly to proper line and grade, that they do not spring or settle, and that they are not removed before the concrete is sufficiently set.

See that back-filling is placed and tamped as required by the specifications.

Keep a record of all extra work, of foundations, of sheeting left in the trench, and of other work that cannot be measured after the completion of the sewer.

See that no ground water flows over concrete or brickwork, or through the pipe laid, except as may be allowed by the engineer.

To this list might be added the making of a detailed daily report to the engineer of all work done, preferably on blank forms furnished for the purpose.

ESTIMATES OF MATERIAL AND COST

GENERAL PRINCIPLES

40. Estimates of Cost.—The engineer frequently has to prepare estimates of the cost of the work, and he should prepare for the guidance of the bidders, schedules of labor and material requird. This is a comparatively easy task in so far as the work directly contemplated is concerned, but collateral work, such as taking care of streams and other sewers and pipes, pumping water, building temporary structures, etc., often is uncertain in amount. In preparing such schedules of information for the bidders, the engineer should remember that reasonable care is necessary, and that his employer will be liable for wilfully mis-

leading statements as regards quantity or quality of work, if because of such statements the contractor suffers loss.

It is required by law in most places that the engineer prepare an estimate of cost on all city work. Such estimates are necessary in order that the city may know how much work can be done for the money on hand, how to finance the work, and whether the proposed bids are reasonable. Accurate estimates of cost also help the contractor in making up his bid.

41. Cost Keeping.—There are several kinds of cost keeping, depending on the ends in view. The engineer and the contractor will want to know the final unit costs on the work, that is, how much it costs to lay 12-inch sewers, how much to build reinforced concrete sewers, etc. Still more important to the contractor is an approximate daily record of the cost of the more important items of the work. When the job is completed, it is too late for the contractor to know that he has been losing money on certain items. He should know these costs every day, so that, where he is losing money, he can take steps to lessen the cost. The engineer is usually called upon to make monthly or progress estimates of the work done, to be used in paying the contractor; when the work has been completed, he will have to make a final estimate.

42. Items of Cost.—The general items of cost include labor, materials, engineering and overhead, profit, plant, and supplies. For the purpose of estimating costs, these items may be divided and subdivided. For instance, one of the divisions of labor may be concreting; and under concreting, the subdivisions may include making, placing, and removing the forms, mixing and placing the concrete, etc. In this way, detailed costs are made available for the more important task of obtaining the unit costs of the work.

Certain basic information in regard to the items entering into the cost of construction will be given in the following articles, with the idea of illustrating some of the points discussed in the articles just previous. The estimating of the material needed, the items to be covered in the work, etc., will be discussed, and

some information given that will be of value in the actual estimation of costs.

43. Methods of Estimating.—Cost estimating methods may be grouped under two general classes: (1) Estimates based on the final cost of completed works of similar character, with proper allowances for local conditions; (2) estimates based on the figures for all the items entering into or making up the structure.

In the first method, total costs are expressed as so much per million gallons of sewage to be treated, the cost of filters per

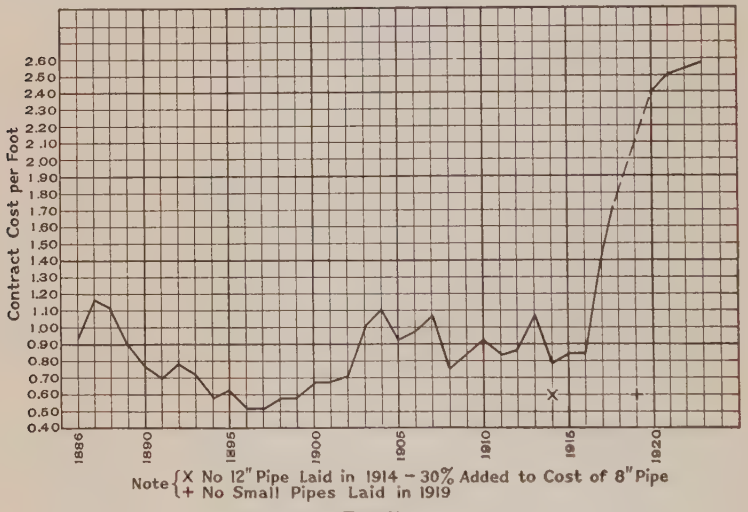

FIG. 30

acre, etc. It is also possible to divide the work into a few major items, such as sewer in place, engineering and overhead, manholes and other appurtenances, etc. In following this method, the engineer must use figures only from cases similar in scope and type, and allowances should be made for local conditions that might affect the cost.

In the second method, the various items of work making up the project, such as excavation, laying pipe, forms, cement and aggregate, mixing, handling, and placing the concrete, etc., are tabulated, and to each item is assigned a unit cost based on

the records of various works of similar character, with such corrections and changes as the judgment of the estimator dictates. The sum of all these costs, with allowances for engineering, administration, and contingencies, is the estimated cost. The greatest danger in this method is the liability of overlooking unforeseen and unforeseeable items.

44. Variation in Costs.—Fig. 30, taken from *Public Works,* illustrates very well the variation in costs from year to year for work of the same character. When it is considered that work varies a great deal from job to job, the difficulty of making fairly accurate estimates of the cost of proposed work is readily understood. The chart, which was prepared by Edwin S. Rankin, engineer of sewers at Newark, New Jersey, shows the fluctuation in the average cost of constructing pipe sewers in that city from 1886 to 1923. In compiling the data for the diagram, 12-inch pipe sewers laid at a depth of 8 to 10 feet were used as a basis. The chart shows a variation of cost from about 52 cents per foot in 1896 to more than $2.50 per foot in 1923.

ESTIMATES FOR PIPE SEWERS

45. Length of Pipe Sewers.—The number of linear feet of pipe sewer of various sizes can be determined approximately from the map. It is not safe, however, to use any scaled distances for estimates, preliminary or other; the exact distances must be taken from the notes, if they are not marked on the map. The map shows the location of the different sizes. From the lengths of the sewer lines must be deducted the space taken out by manholes and branches. The number of Y and T branches depends on the number of house connections, which must be determined. Usually there is one branch for each house on each side, and one also for each vacant lot for improved areas. Branches should be assumed on both sides at distances equal to the customary width of lots, which may vary from 20 to 60 feet according to the locality. A certain allowance (1 or 2 per cent., depending on the locality and the condition of roads) should be added for breakage, but the exact lengths should be stated for contract purposes.

46. Cost of Sewer Pipe.—Sewer pipe is listed at uniform prices by an association of factories, but from these list prices large discounts are allowed. In computing the cost of pipe, it should be remembered that pipe usually has to be cut at a manhole; in estimating the amount of pipe, therefore, every such fractional length should be counted as a full length. To the actual cost of the pipe should be added the costs of transportation, handling, storage, and interest.

TABLE I

LENGTH OF 3-FOOT PIPE SEWER THAT CAN BE LAID WITH 1 BARREL OF CEMENT

Diameter, in Inches	6	8	10	12	15	18	21	24
Length, in feet, 1:1 mortar	840	470	315	210	130	90	70	50
Length, in feet, 1:2 mortar	1,200	675	450	300	190	130	100	70

47. Cost of Laying Pipe.—The cost of laying sewer pipe naturally increases with the size of the pipe. It consists principally of the labor expense, which depends largely on the character of the soil. In dry, firm soil, the cost of laying the pipe is much less than in wet and soft material.

48. Amount of Cement for Laying Pipe Sewers.—Table I gives the approximate length of 3-foot pipe that, under ordinary conditions, can be laid with one barrel (4 sacks) of cement.

ESTIMATES FOR CONCRETE AND BRICK SEWERS

49. Cost of Concrete Sewers.—Where circular concrete sewers are designed with a crown thickness in inches equal to the diameter in feet plus one, an invert thickness equal to the crown thickness plus one, and a haunch thickness equal to $2\frac{1}{2}$ times the crown thickness, the volume of concrete per 100 feet of sewer may be found by the formula

$$Q = 2\,d^2 + 2.5\,d$$

in which Q = number of cubic yards of concrete for 100 feet of sewer;

d = inside diameter of sewer, in feet.

From 5 to 10 per cent. additional should be allowed for waste; and the cost of the forms, as well as the cost of the labor for setting them up and taking them down, should be added.

50. Items of Expense for Brick Sewers.—The cost of brick sewers will include the cost of brick, sand, and cement; the cost of constructing, placing, and moving the templets and centers; the cost of laying the bricks, etc. Other items are, of

TABLE II

NUMBER OF BRICKS PER LINEAR FOOT IN BRICK SEWERS

Circular Sewers			Egg-Shaped Sewers		
Diameter Inches	Single Ring	Double Ring	Horizontal Diameter Inches	Single Ring	Double Ring
24	48	114	24	60	142
27	54	126			
30	60	138	30	75	170
33	66	150			
36	72	162	36	90	198
42	83	184	42	105	225
48	95	207	48	119	253
54	107	229	54	132	280
60	119	251	60	146	308
66		273	66		335
72		295	72		363
78		317	78		390
84		338	84		418
90		359	90		445
96		380	96		472

course, excavation, hauling, storage, etc. The cost of laying the bricks depends on the local wages of the brick masons, the weather, the progress of the job, and many other factors.

51. Number of Bricks.—Table II gives the approximate number of bricks per linear foot necessary to build brick sewers of various sizes. The number given is for 2¼-inch bricks.

52. Amount of Mortar for Brick Sewers.—Table III gives the amount of mortar per linear foot necessary to lay

TABLE III

CUBIC FEET OF MORTAR PER LINEAR FOOT IN BRICK SEWERS

Circular Sewers			Egg-Shaped Sewers		
Diameter Inches	Single Ring	Double Ring	Horizontal Diameter Inches	Single Ring	Double Ring
24	1.00	2.20	24	0.90	1.90
27	1.04	2.31			
30	1.10	2.43	30	1.02	2.12
33	1.17	2.55			
36	1.24	2.70	36	1.15	2.35
42	1.38	2.95	42	1.27	2.57
48	1.52	3.20	48	1.40	2.80
54	1.66	3.50	54	1.50	3.05
60	1.80	3.80	60	1.60	3.30

brick sewers of various sizes; this amount includes a plaster coat on the outside $\frac{1}{4}$ inch thick. Table IV gives the amount of sand

TABLE IV

CEMENT AND SAND IN 1 CUBIC YARD OF MORTAR

Proportion of Cement to Sand	1:1	1:1½	1:2	1:2½	1:3
Bags of cement.....	16.00	12.96	10.92	9.44	8.32
Cubic yards of sand.....	0.6	0.7	0.8	0.9	1.0

and cement required to make 1 cubic yard of mortar for the proportions given. The mix generally used is 1:2.

OTHER ITEMS OF COST

53. Cost of Earthwork.—The cost of trenching is the most uncertain item in estimating the cost of a sewer. The character of the soil so affects the cost of digging that, for the same size of trench, the cost in one soil may be ten times as much as in another soil. Excavation costs are also greatly influenced by the depth of cut. If a laborer can handle dirt at a cost of 20 cents per cubic yard, it follows that, while this figure may be used in trenches less than 6 feet deep, trenching at depths from 6 to 8 feet will cost 40 cents per cubic yard on account of the necessity of rehandling the dirt. In trenches from 8 to 12 feet deep, the cost will be 60 cents per cubic yard, for the dirt will have to be moved three times. Sheeting and bracing is sometimes necessary and is sometimes not needed. The cost of back filling depends on the thoroughness with which tamping and puddling are done.

The width of trench is estimated as follows: For 6-inch pipe to 15-inch pipe, 3 feet; for 18-inch to 24-inch pipe, 4 feet. These values are only approximate, since a trench 6 feet deep for a 6-inch pipe may be only 2 feet wide, and a 25-foot trench for a 24-inch pipe, if sheeting is necessary, must be made about 6 feet wide at the top.

54. Cost of Manholes.—The number of bricks required for manholes is shown in Table V, the bottom not being included.

TABLE V

NUMBER OF BRICKS IN MANHOLES WITH 8-INCH WALLS

Depth, Feet	6	7	8	9	10	11	12	13	14	15	16
Number of bricks	1,100	1,250	1,400	1,550	1,700	1,850	2,000	2,200	2,350	2,500	2,650

A load of gravel and a barrel of cement will make a concrete bottom. If the bottom is of brick, 200 bricks with two bags of cement and two barrows of sand for mortar will be needed.

It has already been explained that the weight of the cast-iron frame and cover for a manhole is about 350 pounds. The cost of these must be added to the cost of the bottom and walls to get the total manhole cost.

SEWAGE TREATMENT

PRELIMINARY CONSIDERATIONS

COMPOSITION AND DECOMPOSITION OF SEWAGE

SOLIDS IN SEWAGE

1. Need for Treatment of Sewage.—Sewage is, perhaps, the most objectionable and dangerous waste product of modern life. Though carried for many miles through sewers, it still retains its power to cause nuisance and disease, and both public health and public comfort demand that it be so treated as to eliminate or reduce to a satisfactory degree these dangerous and objectionable qualities before it is discharged from the sewerage system. Eventually sewage will find its way to a watercourse. If the stream is small, untreated sewage will render the water offensive to sight and smell; floating matter may strand on the banks, thus giving additional opportunity for the spread of disease by flies; fish may be killed, and the stream so contaminated that it is unfit for domestic or community use for many miles below the point of discharge of the sewer. Even in large streams, contamination of the water may extend for many miles from the sewer outlet.

2. Organic and Inorganic Matter.—Sewage is a mixture of water and solid material, such as feces, paper, matches, bits of household garbage, and grease. The solids in sewage consist of both *organic matter*, or substances of which living bodies are composed, and *inorganic matter*, or matter which exists out-

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side of living bodies and which is not necessarily a constituent of living bodies. The organic matter in sewage is both animal and vegetable in character, and includes feces, urea, proteins, carbohydrates, fats, and soaps. It is very unstable and gradually passes through a process of decomposition into other forms. The simple substances of which organic matter is composed are found differently combined in inorganic matter. Inorganic matter in sewage, commonly designated as *mineral matter,* includes chiefly clay, sand, iron, aluminum, and chlorides, carbonates, sulphates, and phosphates of the alkalies and alkaline-earth metals.

Organic matter in sewage is derived from various sources. Though all is not harmful, that which originates with animal life, and particularly with the wastes from human life, is very dangerous, for it may contain germs of disease; and even after a marked degree of purification has been effected, an effluent from such sewage is still potentially dangerous. Since nuisance arises from the decomposition of vegetable organic matter, as well as animal matter, the treatment of all organic matter is necessary.

3. Proportion of Solid Matter in Sewage.—Under usual conditions, sewage consists of not more than one or two parts of solid material to a thousand parts of water, but even this small amount renders the entire mixture objectionable and dangerous. Domestic sewage ordinarily contains from 200 to 2,000 parts or more of solid matter per million, approximately one-half of which is organic and subject to decay. About three-fifths of the organic matter is in solution, the remaining two-fifths being in suspension. Not all sewage, of course, has the same composition and strength. The strength, or *concentration,* will vary with the water consumption, and the composition with the industrial wastes added to the sewage, and with other factors.

4. Variations in Character and Amount of Sewage.—There is considerable variation in the quantity and quality of sewage as it is ordinarily discharged at various hours during the day and night. During the early hours of the day, as from 2 to

5 o'clock in the morning, the flow of sewage is least in volume, and the sewage is weakest as well. From 5 o'clock on, the sewage increases in strength and volume. The flow is usually still greater from 8 o'clock in the morning until 8 in the evening, after which hour there is a decrease. Slight increases are noted in most cases at 9 o'clock in the morning and at 2 and 6 in the afternoon. There also may be variations from day to day. Mondays and Saturdays are always days of heavy flow, and Sunday is always a day of low flow and strength. This is due to the fact that not only is usual activity at a minimum on Sunday, but also few industrial wastes are being discharged. Seasonal variations are also common, as there is greater leakage into the sewers during the wet seasons of the year, and householders allow the water to run in cold weather to prevent it from freezing in their fixtures.

5. Industrial Wastes.—Various industries contribute to the sewers large quantities of wastes. In large cities the proportion of these wastes to the total amount of sewage is small; in a few places industrial wastes form a large part of the volume of sewage. In designing a sewage treatment plant, a thorough investigation should be made to determine the possibilities of trouble from industrial wastes, present or future.

DECOMPOSITION OF SEWAGE

6. Progressive Changes in Sewage.—As already stated, the organic matter in sewage is an unstable product. Consequently, changes brought about by chemical and bacterial actions are constantly taking place in the sewage, tending toward the complete oxidation or reduction of the organic matter. These changes are so rapid that some action often takes place while the sewage is still in the sewers in transit to the treatment plant or point of disposal. Samples taken at points far apart along an outfall sewer will show distinct progress in the process of reduction.

Before the process of decomposition has progressed far enough to use up an appreciable amount of the oxygen orig-

inally present in the sewage, the sewage is termed *fresh*. Under ordinary conditions, sewage absorbs very little oxygen from the air, while the process of decomposition carried on by bacteria exhausts, or nearly exhausts, within a few hours, all of the oxygen of the sewage. The sewage is then said to be *stale*. As the process of decomposition continues after the oxygen disappears, the sewage becomes dark and odorous. This part of the decomposition process is known as *putrefaction*, and the sewage in this stage is called *septic*.

7. Bacterial Action in Decomposition.—In the first stages of decomposition, certain bacteria break up the proteins, starches, sugars, and other organic compounds into simpler substances. In this process, oxidation also occurs; methane, carbon dioxide, hydrogen sulphide, nitrogen, and other gases are given off, and oxygen disappears.

After the oxygen originally available is exhausted, other bacteria complete the process of oxidation by breaking up the compounds and substances resulting from the previous action and changing them into stable forms.

8. Nitrogen Cycle.—During the processes of decomposition just described, the organic matter in sewage is broken up, and the nitrogen contained in it is released. This nitrogen appears first as free ammonia, but through the combination with oxygen which takes place under conditions of ample oxygen supply, the ammonia is changed to nitrites and then oxidized still further to nitrates. Nitrates are necessary to plant life, and plants take them up; animals eat plants, converting them into animal tissue, or the plants die and decay. The animals also die, and their tissues decompose, producing nitrogen. Thus through death and decomposition nitrogen is again formed, and the so-called *nitrogen cycle* is complete.

There are, however, leaks and deviations in this cycle. Certain types of bacteria reduce nitrates to nitrogen gas; this process is termed *denitrification*. Such loss of nitrogen, which after a long period might seriously deplete the earth's supply, is compensated for by the activities of other bacteria that extract nitrogen from the air; this process is called *nitrogen fixation*.

DISSEMINATION OF DISEASE BY SEWAGE

9. Bacteria in Sewage.—*Bacteria* are microscopic organisms belonging to the vegetable kingdom. They have power for both evil and good, as certain types may produce disease, whereas others may aid in the reducing processes whereby the unstable compounds in sewage are changed to stable forms that are unable to cause nuisance. So far as sewage treatment is concerned, the family and history of most bacteria have no special significance. It is mainly necessary to understand the conditions under which the useful bacteria best accomplish their work so that the most favorable environment can be provided for them. Hence, they are usually classed merely according to their mode of living and according to the work they do.

10. Classification of Bacteria.—According to their mode of living, bacteria are classified as *aerobic, anaerobic,* and *facultative*. Aerobic bacteria live and multiply only in the presence of air or free oxygen. Anaerobic bacteria thrive only in the absence of oxygen. Facultative bacteria can live and thrive under either condition.

Bacteria are also divided into *saprophytic* and *parasitic* bacteria. Saprophytic bacteria are refuse eaters, obtain their nutriment from dead organic matter, exist independently of a living body, and are generally harmless to living organisms. These bacteria oxidize organic matter and reduce it to harmless and stable forms. They are, therefore, of great aid and importance in the process of sewage purification. Parasitic bacteria live on or in some other organism, from which they derive nourishment, and cannot live independently of living bodies, except for short periods. They are of importance because some varieties of them are the cause of many diseases.

Parasitic bacteria are further classified as *pathogenic* and *non-pathogenic*. Pathogenic bacteria are those that produce disease in man or other animals. Non-pathogenic bacteria, though living as parasites, do not produce disease.

11. Infectious Diseases.—When pathogenic bacteria penetrate into living organisms and grow and multiply therein, they

produce disease, the nature of which depends on the species of bacteria. Diseases caused by bacteria are called infectious diseases. Pathogenic bacteria may be taken into the system with food or drink, with the air, or through breaks or abrasions in the skin. The methods of infection most important in connection with the subject of the disposal of sewage are through food and drink, as these are the most important means of spread of pathogenic bacteria from sewage.

12. Filth-Borne Diseases.—The most important diseases due to, or spread by, improper disposal of sewage include typhoid fever, cholera, diarrhea, colitis, commonly called summer complaint of infants, enteritis, and hookworm. They are generally grouped under the designation of filth-borne diseases, and, excepting hookworm, can be contracted only by swallowing human excreta or material contaminated with them. Of course the specific germ of the disease must be present, but these germs come only from the bodily discharges of persons who are suffering, or have previously suffered, from the disease.

In the United States the specific germs of cholera are not found, or are found so rarely as to be of no importance from a public-health standpoint. Typhoid fever, however, is a common disease; about 10,000 people die every year from typhoid in the United States, and ten times this number suffer from it, but recover. It is caused by a microscopic organism, *Bacillus typhosus,* which attacks the intestines.

13. Methods of Spread of Bacteria.—The spread of typhoid is typical of the spread of other filth-borne diseases. The two most common methods are probably through flies and water. Flies are important mainly where there are no sewers, and where open toilets are used. Disease may also be caused by flies carrying germs from stranded flotage or filth from sewage discharged into streams. Where sewers exist, distribution of disease is more often through the agency of an infected water supply.

There are two main methods of spread of water-carried infection; namely, over the ground surface, and underground.

The former is undoubtedly the most important, for it includes the carriage of contamination by streams, flood waters, and rains. Underground spread of disease has the greatest appeal to the imagination, but is of little, if any, importance except in limestone sections, where water may travel for miles through crevices or channels in the rock. Ordinarily, water is sufficiently purified by passage at low velocities through soil for a distance of two or three hundred feet. The spread of disease over the ground surface is illustrated by an outbreak of typhoid fever at Plymouth, Pennsylvania, during which more than 1,000 cases developed in 10 days in a population of about 8,000. The outbreak was traced to infection from the excreta of a single patient living on the watershed of the stream from which the village drew its water supply. The typhoid germs retained their vitality until the spring thaws, when they were washed into the stream and thence found their way into the public water supply. Many other outbreaks have been reported as being caused in a similar manner.

Cases of typhoid contracted from eating oysters or shellfish taken from waters contaminated with sewage are not uncommon. An epidemic from this cause occurred in Chicago, Illinois, in the winter of 1924–1925.

14. Life of Pathogenic Bacteria.—Some of the pathogenic bacteria known to science have extraordinary powers of resistance, and will retain their capacity to cause disease even after long periods. Fortunately, none of the disease germs ordinarily carried by sewage is so resistant. Under favorable conditions, the typhoid bacillus may retain its vitality for 2 or 3 months, but under most conditions the life is much shorter. In sewage effluent, after the organic matter has been decomposed, typhoid bacilli rarely live longer than 10 days or 2 weeks; in soil, under most favorable conditions, they may survive as long as 100 days; in water the life period is not so great as in soil, but during their life period the germs may be transported for miles by running streams. In general, typhoid bacilli do not appear to multiply to any extent in either water or sewage, but instead decrease rather rapidly in number. In ice, typhoid

germs will live in slowly reducing numbers for a considerable length of time, probably losing gradually the power to cause disease.

SEWAGE ANALYSES

15. Value of Analyses.—Chemical analyses are of great value in the treatment of sewage. The determination of the content of sewage in regard to nitrogen, sulphur, potassium, ammonia, and many other elements may furnish important information relative to the best method of treatment, or may determine the amount of treatment necessary for satisfactory results. It is even more important to know the amount and condition of the organic matter, and the changes produced in it by various processes of treatment, in order to determine the effect that may be produced in rivers or streams by the admixture of raw or partially treated sewage.

16. Data Obtained by Analyses.—Analyses of sewage generally show the total dry solids, the proportion of these that are combustible, usually referred to as *loss on ignition,* and the proportion of ash remaining; also the amount of albuminoid and free ammonia, of chlorine or chlorides, and of nitrites and nitrates; and the amount of oxygen required to oxidize the organic matter, sometimes called *oxygen consumed*. Analyses may be made for the number of bacteria per cubic centimeter and the kinds of bacteria. However, these determinations are not very important in raw sewage, for untreated sewage contains so many and such different kinds of bacteria that the presence of most kinds can be taken for granted. When needed, a bacterial analysis of sewage is made in the same way as the purity of water is tested.

17. Total Dry Solids.—The dry solids in sewage indicate, to a considerable degree, the concentration or strength of the sewage, but the proportions of the organic and mineral matters included in the solids may vary widely. The amount of total solids in sewage is generally reduced materially by treatment.

18. Loss on Ignition.—The combustible solids measure the amount of organic matter in the sewage, but no distinction

is made between that of animal origin and that of vegetable origin. The proportion of combustible matter in sewage is also materially reduced during the course of treatment.

19. Fixed Residue, or Ash.—The residue, or ash, left after combustion represents approximately the quantity of mineral matter contained in the sewage, which is of importance because it affects the amount of sediment that may be obtained from the sewage after treatment. The quantity of ash does not indicate exactly the amount of inorganic matter on account of the volatilization of some of the inorganic compounds originally present.

20. Organic Nitrogen; Albuminoid Ammonia.—*Organic nitrogen* is the nitrogen combined with other elements in the form of organic matter. Compounds of organic nitrogen in sewage are divided into two classes, which may be called albuminoid ammonia and residual organic nitrogen. Albuminoid ammonia does not exist free in sewage, but is derived in the analysis by decomposing the organic nitrogenous substances. In the process of sewage treatment, albuminoid ammonia and residual organic nitrogen are broken up and converted into nitrates. The determination of the two kinds of organic nitrogen is valuable as a means of indicating the degree to which oxidation of the organic matter has progressed.

21. Free Ammonia.—Free ammonia is derived mainly from organic nitrogen. It marks the first distinct step in the process of decomposition, and, as the process of stabilizing the sewage continues, ammonia finally changes into nitrates. The quantity of ammonia present in sewage is a valuable indication of the freshness of the sewage.

22. Chlorides.—Chlorides are a most persistent constituent of urine, both animal and human. Consequently, the quantity of chlorides present in sewage is an indication of the concentration of the sewage.

23. Nitrites and Nitrates.—Nitrites and nitrates are formed by the combination of oxygen with nitrogen derived

from the breaking up of organic substances. The complete reduction of the organic compounds and the combination of their nitrogen with oxygen in the form of nitrates indicates the final stage of purification; hence the progress in purification is measured by the increase in nitrates. Table I shows the reduction of organic nitrogen and the increase in nitrates accomplished by a sand filter in the process of treatment.

TABLE I

LOSS OF ORGANIC MATTER IN SAND FILTER

(*Parts in 100,000*)

Samples Analyzed	Organic		Intermediate	Inorganic
	Free Ammonia	Albuminoid Ammonia	Nitrites	Nitrates
Sewage.....	1.989	.843	.0001	.000
Treated effluent....	.004	.010	.0003	1.880

24. Oxygen Consumed.—The ultimate result sought in the treatment of sewage, as far as chemical analyses are indications of purity, is the complete oxidation of all organic matter; and so the amount of oxygen necessary to accomplish this is an indication of the degree to which purification has been carried, and of the amount of impurities still in the sewage. The quantity of oxygen necessary is determined experimentally by adding to a definite amount of sewage certain compounds of known composition, which will lose oxygen that may be necessary to oxidize the organic matter. The process mainly measures the quantity of carbonaceous matter in the sewage.

25. Standard Methods of Analysis.—The American Public Health Association has published a booklet entitled *Standard Methods of Water Analysis,* which is considered the authority on methods of analysis for water and sewage. According to this standard, the total amount of dry solids may first be determined by evaporating the liquids and weighing the residue. Then, by heating the solid residue to a low red heat,

the organic matter is consumed, and the remainder is the inorganic matter. The percentage of the total solids that are inorganic is found by dividing the weight of the inorganic remainder by the weight of the total solids and multiplying the quotient by 100; and the percentage of the total solids that are organic is computed by subtracting the percentage of inorganic matter from 100. The amount of material in suspension in the sewage can be determined by filtering the raw sewage. Free ammonia may be driven off simply by boiling the sewage, and the amount determined by taking the difference between the weight of the original sample and the weight of the remainder. Albuminoid ammonia is measured by the amount of permanganate of potash required to oxidize it. The methods of testing for chlorides, nitrites, and nitrates belong to the field of chemistry and will not be taken up here.

METHODS OF TREATING SEWAGE

INTRODUCTORY EXPLANATIONS

26. Sewage Disposal and Treatment.—The entire process of getting rid of sewage is called *sewage disposal,* while *sewage treatment* includes only the steps and processes by which sewage is transformed into a liquid that, under the conditions prevailing in the locality, will meet the demands of sanitation, health, and decency. Sewage is very rarely purified, according to the commonly accepted meaning of the word, and, therefore, the term sewage purification is often misleading when sewage treatment is meant.

27. Objects of Sewage Treatment.—There are two main objects to be accomplished by the treatment of sewage. The first is the elimination of disease-bearing bacteria to such an extent as may be demanded by local health conditions; the second is the stabilization of the sewage without producing nuisance or odor, or endangering health. To accomplish these objects the organic matter must be taken out and rendered

harmless by decomposition and oxidation, and the water, which makes up such a large part of the volume of sewage, must be restored to a degree of purity determined by local conditions.

28. Impurities To Be Treated.—Specifically, the matter entering into the composition of sewage that has to be dealt with in its treatment includes solid feces, urine, household wastes, some grit and street washings, and industrial wastes.

Solid feces are usually handled without difficulty by the process of sewage treatment. Urine is the main source of ammonia, through the fermentation of the urea. The volume of urine excreted daily is from 10 to 15 times as great as the amount of feces, and the total solid content is greater in urine than in feces. The larger solids of household wastes are disposed of as garbage, but wash water, vegetable refuse, fragments of food, and wastes from laundry and bath tubs, kitchen sinks, and other connections contribute to the volume of sewage. These household wastes form a large proportion of the sewage of a city; in bulk, they may amount to one-half of the flow. Moreover, the dish water and sink wastes contain so much grease that they contribute that part of the sewage which is, perhaps, the most difficult to treat successfully.

Grit and street washings do not present difficult problems. In fact, in separate sewerage systems they will not appear in appreciable amounts except in unusual cases. Industrial wastes are often produced in large quantities, and may interfere with the processes of sewage treatment. It is customary, and any city has ample power, to require that, where necessary, the manufacturing wastes be treated before they are discharged into the sewers.

29. Steps in Treatment of Sewage.—Sewage treatment may be said to be carried on in two steps, primary and secondary. In the *primary treatment* the grosser solids are removed from the sewage and the more complex compounds in the sewage are decomposed and broken up. These processes, however, do not carry the sewage to that point of purification which is often required to prevent nuisance and odor, and, hence, secondary treatment is needed. In the course of *secondary*

treatment the sewage is oxidized to stable forms that will not further decompose with odor, and the number of bacteria is greatly reduced. The process of decomposition is generally accomplished by anaerobic bacteria, and oxidation by aerobic action.

The process of sewage treatment may also be considered as taking place in four steps. In *Home Sewage Disposal,* by Hardenbergh, these steps are listed as follows:

1. Preliminary action of decomposition while the sewage is in transit to the plant.
2. Anaerobic decomposition and clarification which takes place in the sewage and is generally termed primary treatment.
3. Semianaerobic breaking down of the substances derived in step 2.
4. Secondary treatment, or nitrification and complete aerobic oxidation to gases, nitrates, and carbonaceous residues.

30. Outline of Methods of Treatment.—The particular method of treating sewage to be adopted will depend almost entirely on local conditions. It may consist merely of *dilution,* or the discharge of the sewage into a body of water. Where dilution is not possible, a different method of treatment is necessary. The other methods in more common use accomplish results through biologic means. Primary treatment includes *screening,* or the removal of a portion of the coarser solids by passing the sewage through screens; and *sedimentation,* or the passage of the sewage through tanks, either with or without dosage with chemicals, so that the larger and heavier particles of sewage settle out from the liquid. Secondary treatment may be given by *filter beds,* which strain out the solid matter and also effect bacterial purification; by *contact beds,* in which sewage is allowed to stand for some time in contact with stone, slate, coke or other substances; by *irrigation,* or the distribution of sewage over large areas of land in such a fashion that the soil absorbs the sewage; and by *disinfection,* or the killing of the bacteria in sewage by means of chemicals.

DILUTION

31. Meaning and Importance of Dilution.—Dilution is the disposal of sewage by discharging it untreated or partly treated into a stream or other body of water, in such a manner and under such conditions that it can no longer be offensive or a serious menace to health. As indicating the importance of dilution as a method of disposal, it has been estimated that over 80 per cent. of the people in the United States served by sewerage systems dispose of their sewage by dilution. This high percentage is accounted for largely by the fact that nearly all the big cities are located on lakes, rivers, or oceans, where dilution is the natural method of disposal.

32. Conditions for Successful Disposal by Dilution. Metcalf and Eddy give the following conditions as essential or favorable to successful disposal by dilution: (*a*) freshness of sewage; (*b*) freedom of sewage from floating matter and solids capable of settling; (*c*) thorough diffusion, or mixing, through the diluting water; (*d*) diluting water of high oxygen content; (*e*) swift currents to carry the sewage to points of unlimited dilution; (*f*) biological equilbrium, or non-putrefying condition of solid matter; (*g*) absence of quiet places tending to facilitate sedimentation accompanied by sludge deposits. Most important of all is ample dilution; there should be at least 40 parts of water by volume to each part of sewage.

33. Preliminary Treatment.—Preliminary treatment of the sewage may or may not be necessary, depending on the degree of dilution that may be attained. If the flow of water is ample to dispose of the sewage, no treatment may be necessary. On the other hand, where dilution is not ample or where conditions for this method of disposal are not favorable, a degree of treatment dependent on local conditions must be provided, in order to prevent nuisance. If decomposition has progressed so far that the sewage has lost all of its oxygen, there will have to be an extra amount of oxygen available in the diluting water, which means a greater volume of water and more thorough

mixing. Otherwise it will be necessary to treat the sewage before discharge so as to decrease its demand for oxygen.

Where considerable solid matter is carried by the sewage, or where the conditions may favor settlement or stranding of these solids, it may be necessary to give the sewage such preliminary treatment as will remove this objectionable matter. In most cases this is accomplished by straining or screening the grosser solids from the sewage, or by sedimentation, which will remove a considerable part of the suspended solids.

34. Mixing Sewage With Water.—Thorough diffusion of the sewage through the water is necessary for proper disposal by dilution. Swift currents are a great aid in this, for they assist in spreading the sewage and in transporting it to other places where additional dilution is available. In some of the more recent treatment works, the sewage is discharged through a number of orifices. Where the sewage is discharged into tidal waters, studies should be made of the directions of the currents so as to locate the point of discharge that is most suitable for proper dilution, and most likely to prevent stranding of solids on the shores. In some places tanks are employed to hold the sewage for discharge when the tide is flowing out.

35. Other Conditions.—If the water is of high oxygen content, disposal by dilution is more likely to be accomplished without difficulty. If the water is low in oxygen, a greater degree of dilution is necessary. In those cases where suspended solids are not removed, sedimentation may occur where the velocity of the stream drops below one foot per second; therefore, the point of disposal should be so chosen that there will be small opportunity for sedimentation to occur.

36. Effect of Dilution on Organic Matter and Bacteria. If there is plenty of oxygen in the diluting water, the aerobic bacteria predominate, and the organic matter is converted into stable forms by the process of oxidation. Where dilution is ample, the number of bacteria decreases rapidly, mainly because of the ample supply of oxygen and the lack of food supply. On the other hand, if dilution is inadequate, and there remains

plenty of food, bacteria may thrive under favorable conditions to such an extent as to exhaust the oxygen supply of the water, whereupon nuisance and odor occur.

SCREENING

37. Suspended Matter.—Sewage often contains many particles of suspended matter, large and small, organic and inorganic. Among these are sticks, rags, fruit peels, bits of vegetables, bottles, street grit, sand, cinders, and similar material. While all sewage contains more or less material of this sort, sewage from combined sewers is particularly liable to be laden with it.

38. Necessity for Removing Suspended Matter.—It is often desirable or necessary to remove all or part of the larger suspended matter. If the sewage is to be pumped, rags and sticks may clog the pumps; and bottles, grit or cinders may injure them. In the case of tanks treating sewage by methods that require bacterial action, it is very desirable to remove grit, sand, cinders, and rags, which may interfere with such action. Where sewage is discharged into a body of water, it is almost always desirable to remove all solid matter that may form decomposing deposits in the water, or may strand on the shores. This material is often best separated by the use of grit chambers, racks, and screens.

39. Grit Chambers.—Grit chambers are necessary where combined sewers are used, being designed to remove from the sewage as much as possible of the grit, sand, and other inorganic matter, but not the organic material. They accomplish their purpose by decreasing the velocity of the sewage just enough to cause the heavier material, such as sand and grit, to settle out, while the lighter organic material is carried through.

Several problems are involved in the design of grit chambers. It has been found that a velocity of not less than 1 foot per second will give best results in separating the organic from the inorganic material; but even at that velocity, some of the heavier organic solids will settle out. The length of the cham-

ber depends mainly on the time required for the grit to settle, which in turn is dependent on the depth of sewage. It is difficult to give exact figures on the best length for grit chambers, for much depends on the composition of the sewage. The ends of grit chambers should generally be flared so that changes in velocity will be gradual, and currents and eddies avoided. A uniform velocity is desirable, but cannot be maintained because of the large variations in the flow of sewage. Generally two or more separate basins are provided. In periods of storm flow, all basins can be employed; and one or more of them can be taken out of use for dry-weather flow or for cleaning. There is no settled practice in regard to the depth of grit chambers. Some designers use shallow chambers, whereas others prefer a considerable depth.

40. Bar Screens or Racks.—Screens or racks constructed of flat iron bars set on edge may be placed across the channel through which sewage flows. The bars are spaced from $\frac{1}{2}$ to 1 inch on centers, and the width and length of the screens are such as may be necessary to provide for the given flow of sewage. The screen is usually set with the bars sloping in the direction of the flow of sewage, the angle of slope varying from about 30 degrees to 60 degrees. Racks are used mainly to remove from the sewage sticks, rags, or other materials likely to injure or clog pumps or other machinery. They are often used in connection with grit chambers, in which case they may be placed at either end, but generally are located at the inlet end.

41. Fine Screens.—The use of fine screens for the purpose of removing from the sewage a part of the suspended solids is a recognized method of sewage treatment. Screening in the United States is confined mainly to places where the removal of the larger solids from the sewage will meet the local requirements for treatment. The types of screens most generally used are known as the belt, disk, drum, wing, and concentrating screens. The belt screen, shown in Fig. 1 (*a*), is made up of an endless chain of perforated plates. As the chain

moves, the screenings are raised above the sewage. They are then removed by brushes. The disk type of screen is well illustrated by the Riensch-Wurl screen shown in (*b*). It consists of sheets of perforated metal, and is placed in the sewage at an angle, the upper portion being above the level of the sewage. The disk is revolved slowly, and as the screenings are raised they are removed from the screen. A drum screen, shown in (*c*), is a screen in the form of a horizontal cylinder through which the sewage flows. It is revolved at a low rate of speed, the screenings being removed and carried

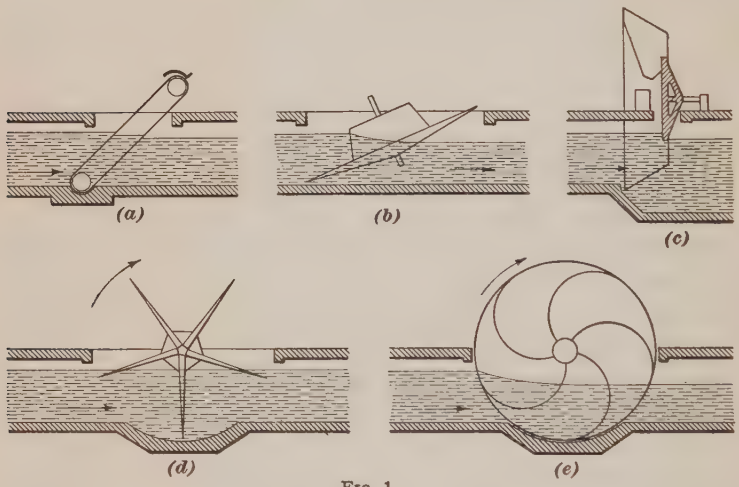


FIG. 1

away so as to keep the screen clean at all times. Some drum screens, such as the Link-Belt type, are longer and have a smaller diameter. The wing screen, shown in (*d*), is mounted on a shaft and has four, five, or six wings or vanes, which are revolved against the flow of sewage. The shovel-vane screen, shown in (*e*), is a variation of the wing screen. The concentrating screen of the Dorr type is a drum screen that revolves at a high rate of speed. By this device a part of the screened sewage is carried above the level of the incoming sewage and the screenings are washed into a pit, from which they are removed by an elevator.

42. Cleaning Screens.—All screens, and especially the finer ones, clog quickly unless cleaned carefully and continuously. To be effective, continuous automatic cleaning must generally be provided for all the screens described in the preceding article. Bar screens may be cleaned by hand or mechanically; hand cleaning is more common, although mechanical cleaning is practiced in large installations. In the former method the screenings are raked off the bars at frequent intervals, and hauled over the top of the rack onto a trough or platform. Mechanical cleaning devices usually consist of rakes driven by chains, shafts or gears, which automatically and continuously clean the screen by raking the screenings into a trough or conveyer.

43. Amount of Screenings.—The amount of solids removed by screens varies with the fineness of the screen, with the freshness of the sewage, and with other factors. Much depends on whether the sewage has been pumped, and on the distance of flow through sewers before it reaches the screen. Both the pumping and the flow through sewers tend to break up the larger solids in the sewage, and thus reduce the amount of matter screened out and increase the tendency of the screen to clog. Stale sewage also tends to clog the screens. In ordinary practice, only the larger solids and floating material are removed by screens.

44. Disposal of Screenings.—Screenings are very offensive and must be disposed of promptly. In some plants the screenings are pressed and burned; at other plants they are buried. Generally their disposal is a costly and disagreeable task.

TANK TREATMENT OF SEWAGE

METHODS OF TANK TREATMENT

45. When sewage is allowed to flow slowly through a tank, the suspended solids in the sewage that are capable of settling under the existing conditions of velocity of current and period of detention, gradually settle out and fall to the bottom, while the partially treated sewage passes out of the tank. The solid material that settles out of sewage during tank treatment is called sludge. In the simplest form of tank treatment, which is called plain sedimentation, the sludge is removed frequently. If, instead of removing the sludge, provision is made to retain it in the bottom of a tank for an indefinite period, during which time it is allowed to undergo anaerobic decomposition, the treatment is called the septic process. In the septic process, the sewage not only has opportunity for sedimentation, but undergoes anaerobic decomposition. In some places, however, it is desirable to allow the sludge to digest or decompose but at the same time to keep the sewage free from the process of anaerobic decomposition. The Imhoff tank, also called the Emscher tank, is designed for this purpose. It is provided with a slot or trap through which the settling solids fall into a chamber beneath. In this chamber the settled solids are allowed to decompose, but without contact with the sewage, which is kept fresh. In the activated sludge process, sewage is held in tanks for several hours in contact with about 20 per cent. of sludge that has previously been produced by this process, meanwhile undergoing agitation by means of air. The result is a rapid clarification, and an oxidation of the organic matter. In treatment by chemical precipitation, sedimentation is aided by adding to the sewage certain chemicals; these produce a floc, which settles rapidly and carries down with it much solid matter.

Second to dilution, tank treatment methods are of the greatest importance in modern sewage treatment. A very great majority of treatment plants include as a primary process some method of sedimentation or tank treatment.

PLAIN SEDIMENTATION

46. Kinds of Tanks.—Inasmuch as sedimentation is aided by a low velocity of flow through a tank, and by the absence of eddies and currents, sedimentation tanks should be so designed as to reduce the velocity and eddies as much as possible.

There are two types of sedimentation tanks, the horizontal flow and the vertical flow. In the former, the sewage flows horizontally through the tank, which is generally rectangular in plan, with a length perhaps two to five times the width, and has a depth of 6 to 10 feet. In the vertical-flow tanks, which were first used in Germany, the sewage passes through a vertical pipe to the bottom of the tank and then flows upwards, the solids meanwhile settling to the bottom.

47. Horizontal-Flow Tanks.—A simple form of horizontal-flow tank is shown in Fig. 2. From the sewer *a*, the sewage enters the inlet trough *b* and then passes through the holes *c*

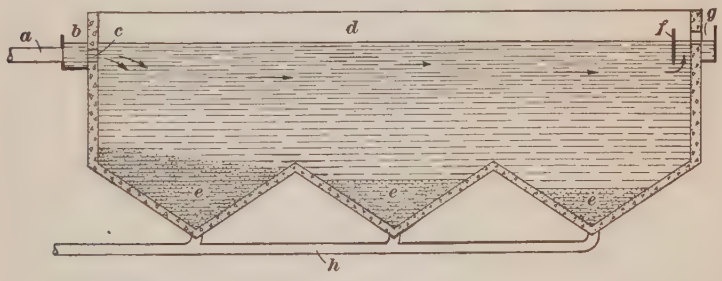


FIG. 2

into the tank *d*. As the sewage flows through the tank, the solid matter settles to the bottom and collects in the hoppers *e*. Discharge of the treated sewage, or *effluent*, takes place under the baffle *f* and over a weir into the outlet channel *g*. The sediment can be removed from the hoppers through the pipe *h* by the hydraulic pressure of the liquid in the tank. The side slopes of these hoppers need not be great; where the increased depth of excavation is costly, the slope may be 1 vertical to 4 horizontal, but a slope of 1 to 1 is better.

Baffles are frequently used at the inlet instead of the holes *c*, but the latter method is somewhat preferable, as it gives a more even distribution of the sewage. In order to prevent cross currents and direct flow, which may allow some of the sewage to flow from the inlet to the outlet without proper sedimentation, baffles or partitions are commonly inserted in the tank at intervals. These should be so placed as to obtain a uniform flow across the entire width of the tank. Too many baffles may constrict the flow so much that they will defeat the pur-

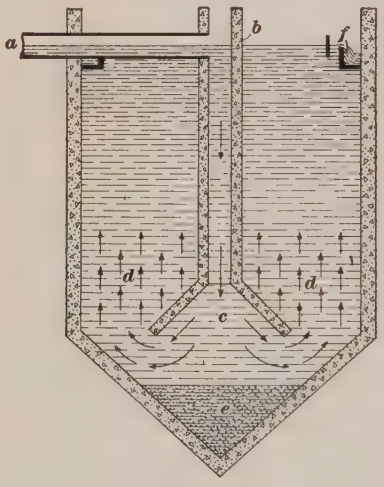


FIG. 3

pose of their construction. The heavier and bulkier solids will settle out first, that is, at the inlet end of the tank. Hence, this part of the tank should be deeper, or else provision should be made to remove the solids more frequently than from the remainder of the tank.

48. Vertical-Flow Tanks.—Vertical-flow tanks are usually made circular in plan, although this type of construction costs more than a rectangular plan. In Fig. 3 is illustrated the essential features of a vertical-flow tank. The sewage enters at *a* and passes through the down pipe *b* into the tank. As it passes out of the funnel *c*, its velocity of flow is reduced so

that at *d* it loses a large part of its suspended matter, which settles to the bottom *c*. The effluent escapes over the overflow weir *f*. The inlet pipe is carried down to a point only a few feet above the bottom, and the sewage leaves this pipe just far enough from the sludge deposits in the bottom to avoid disturbing them. The sewage flows upwards at a gradually decreasing velocity, as the section of the tank increases, and flows out from the top of the tank over the outlet weir. Suspended matter carried by the upward-flowing current gradually loses its velocity and tends to settle through the liquid. As more solids are brought upwards, they are collected and held until the mass becomes solid enough to sink. The depth of vertical-flow tanks varies. Those at Gloversville, New York, are about 45 feet deep.

49. Cleaning Sedimentation Tanks.—The sludge that collects in the bottom of a sedimentation tank must be removed at frequent intervals. In summer, sludge must be removed at least every week, and preferably more often; in winter, the period may be extended to 2 or 3 weeks. In some plants sludge is drawn daily. If the sludge is not removed promptly, the sedimentation tank becomes a septic tank through the anaerobic decomposition of the sludge in the bottom.

50. Size of Tanks.—The size of a sedimentation tank will depend, of course, on the amount of sewage to be handled, and also on the length of time, called the *detention period,* required for the flow of sewage to fill the tank, or to displace a tank full of sewage. For instance, if the flow of sewage is 10,000 gallons an hour, and the detention period is to be 10 hours, the required capacity of the tank is $10 \times 10{,}000 = 100{,}000$ gallons. The detention period chosen will depend on several factors, such as the freshness of the sewage and the degree of treatment necessary or desirable. If possible, actual tests should be made of the sewage to be treated to find the proportion of solids in suspension that will be removed by a sedimentation period of 2, 4, 6, 8 or more hours. Generally, a detention period of 4 to 6 hours is most desirable. The velocity of flow through the tank should not exceed 50 feet per

hour, and may be much less. Flexibility, or the ability to handle decreased or largely increased flows, may be obtained by having several basins so arranged that one or more may be used at any time.

51. Degree of Treatment.—The duty of a sedimentation tank is to remove as much as possible of the material in suspension in the sewage. Although there will be some variations from season to season, a sedimentation tank should remove from 40 to 50 per cent. of the suspended solids. Occasionally a much higher removal is obtained, but such a performance cannot be maintained consistently.

52. Advantage and Disadvantage.—The main advantage of a sedimentation tank over a septic tank is the fresher effluent. This feature may be of considerable value in places where sewage flows through a long network of sewers and reaches the plant in a stale or septic condition. The greatest handicap is the disposal of the sludge, which is fresh, sticky, and foul-smelling.

In comparatively recent years the sludge from sedimentation tanks has been digested in separate tanks, the capacity of which is based on the population contributing sewage to the plant. The clarification of the sewage is the same as that in an Emscher tank; and it is more assured, for the process of sludge digestion cannot interfere with the settling of the sludge. The digested sludge has all the qualities of first-class Emscher-tank sludge.

SEPTIC TANKS

53. Principle of Septic-Tank Treatment.—In sedimentation tanks, the sludge is removed at frequent intervals; in septic tanks, the sediment is allowed to remain in the tank for from 3 to 9 months or more, during which time it undergoes decomposition by the action of anaerobic bacteria. As a result of this action, there is a reduction in volume and a decrease in offensiveness. Since the oxygen in the sewage is rapidly exhausted, however, the effluent from a septic tank has a high oxygen demand.

54. Reduction and Clarification.—By the processes of sedimentation and anaerobic decomposition in a septic tank, the sewage is relieved of a large portion of its objectionable matter. The solids remaining in the tank are changed to more stable forms, while the finer suspended matters and the material in solution are carried out of the tank. Gas is formed by the action in the bottom of the tank, and gas bubbles rise, carrying with them particles of solid material. Some of these particles add to the scum on top of the tank contents, whereas others are caught by the current flowing through the tank, and are carried out.

The result of these processes of sedimentation and decomposition is a partial clarification or removal of some of the more offensive solid materials, and the disappearance of most of the oxygen originally contained in the sewage. The sludge decomposes in the bottom of the tank. Some of the material in the effluent as it passes out of the tank is still putrescible, that is, subject to decay, and in need of further treatment.

55. Detention Period.—In most cases a detention period of 4 to 8 hours gives very good results, but it sometimes is difficult to hold the sewage in a septic tank for this length of time. This is especially the case in large cities where the sewage is stale when it reaches the treatment plant, that is, the process of anerobic decomposition is well under way. Prolonged retention in a septic tank under these conditions may cause disagreeable odors, and makes secondary treatment more difficult.

56. Sludge Volume.—The volume of sludge produced by the septic process will be much less than the amount obtained by plain sedimentation, and the sludge will generally be much easier to handle. A good basis of design is to allow for a deposit of 50 to 60 per cent. of the suspended matter in the sewage, with a possible period of 9 months between cleanings. Owing to the decomposition of the sludge in the tank, there is a considerable reduction in volume. It was at first thought that all solid matter would disappear, and that the tank would never need to be cleaned, but this is not so. The reduction in

the volume of organic matter varies, but averages possibly 25 to 30 per cent.

It should be noted that sludge will usually contain between 90 and 95 per cent. of water. The volume of solids is twice as great in a given quantity of sludge containing 90 per cent. of water as in the same quantity of sludge with 95 per cent. of water, for 90-per-cent. sludge has 10 per cent. of solids whereas 95-per-cent. sludge has only 5 per cent. of solids. Hence, for a given quantity of solid matter, 95-per-cent. sludge occupies just twice as much space as does 90-per-cent. sludge. It is desirable, therefore, to reduce as much as possible both the volume of sludge and the percentage of water in the sludge.

57. Size of Tank.—The factors governing the size of septic tanks are practically the same as those influencing the construction of sedimentation tanks, except that extra space should be provided for the deposit of the sludge, which, in the operation of the septic tank, is allowed to remain for several months. A detention period for the sewage of 4 to 8 hours should be assumed, and additional space provided for a volume of sludge equal to 25 per cent. of the capacity of the tank.

58. Dimensions and Arrangement.—The depth of an ordinary septic tank should be between 8 and 10 feet. Little advantage is obtained from a greater depth, while the cost generally increases rapidly with the depth. The length of a tank is usually from two to five times the width.

It is extremely desirable to have the tank capacity so arranged that it will be most flexible. This object is best obtained by having two, three, four, or even more, tanks so constructed that each of them may be used separately or together with the others. The operator can then take care of reduced or greatly increased flows, and has an opportunity to throw tanks out of service for cleaning or repairs without interfering with treatment processes. The sizes of the individual units will depend on various factors, such as the detention period desired, the character of the sewage, the space for sludge digestion, and topographic conditions.

59. Details of Construction.—Many construction features of a septic tank, such as the inlet, outlet, and baffles, are similar to the parts already described with reference to a sedimentation tank. The bottom in most septic tanks is made flat or nearly so; in some tanks the bottom has a slight slope to aid in removing the sludge. There is no reason, however, why septic-tank bottoms should not be built with hoppers, as in Fig. 2, and with drain pipes so arranged that the sludge can be removed as desired without interfering with the operation of the tank and without drawing off the sewage.

60. Covers for Septic Tanks.—Inasmuch as the processes going on in a septic tank are mainly anaerobic, it was formerly thought that covers to exclude as much as possible of the air were necessary. Experience has demonstrated, however, that so little air can be absorbed by the surface of sewage in a tank that it is immaterial whether or not tanks are covered. It may, however, be desirable to cover a tank to lessen odors.

61. Operation of Septic Tanks.—It is advantageous to keep the contents of a septic tank as quiet as possible. All inlet and outlet channels and weirs should be kept clean and free from accumulations. The scum and sludge should not occupy more than 25 per cent. of the tank capacity; when this condition is reached, the sludge should be removed. In general, it is preferable to remove sludge in the spring and fall; and also at other times if particles of digested sludge appear in the effluent.

When sludge is removed frequently it will contain less organic matter but a greater percentage of water, and, consequently, the volume of sludge for a given amount of organic matter will be greater. On the other hand, the longer the sludge remains in the tank, the more compact it becomes, and the smaller the percentage of water it contains. An apparently small change in the moisture percentage makes a big difference in the volume of sludge. As previously stated, sludge with 95 per cent. of water will occupy twice as much space for the same amount of solid matter as will sludge with 90 per cent. of water.

62. Advantages of Septic Treatment.—There is in a septic tank an average reduction of 25 to 30 per cent. in the volume of solid matter; therefore, less sludge need be handled than when a sedimentation tank is used. Moreover, the removal of a large percentage of suspended matter from the sewage, such as is accomplished by the septic tank, makes easier the processes of secondary treatment, and reduces the area of ground necessary for that purpose. Furthermore, the sludge produced by septic-tank treatment is not very objectionable and is much more easily handled than that produced by screening or plain sedimentation.

IMHOFF, OR EMSCHER, TANKS

63. Features of Operation.—As compared with the types of tanks previously described, two new principles are introduced in the Imhoff or Emscher tanks: (1) The materials in the sewage that will settle or float are separated at once from the remainder of the sewage; (2) the sludge is allowed to decompose under anaerobic conditions without contact with the sewage that is passing through the tank. The elimination of the solids, which comprise a large part of the objectionable matters in the sewage, is a big step forward. The separation of the decomposing sludge from the sewage passing through the tank reduces odors, facilitates the after-treatment of the effluent, and shortens the detention period necessary.

64. Types of Imhoff Tanks.—There are two general types of Imhoff tanks, the horizontal flow and the radial flow. These correspond generally to the two types of sedimentation tanks already described, and the principles involved in each are the same.

65. General Description.—The essential features of an Imhoff tank are shown in Fig. 4. The sewage flows quietly through the settling chamber *a*, where a detention period between 2 and 4 hours is provided. The solids that are capable of settling sink, as the sewage passes through the tank, and slide down the sloping sides of the settling chamber through the

trap or slot *b* into the digestion chamber, or sludge chamber, *c*. When decomposition of these solids take place, the rising bubbles of gas cannot get back through the slot *b* and pass through the incoming liquid, for they are diverted by the overhang of the sloping bottom of the settling chamber. The bubbles, therefore, rise into the gas chamber *d*. As a consequence, the sewage flowing through the tank has better opportunities to

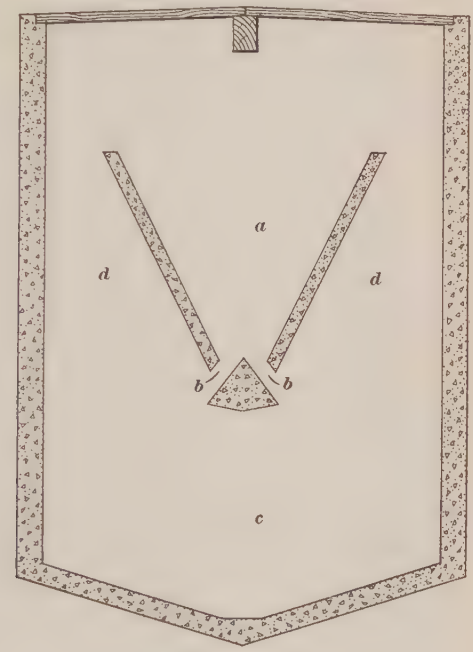


FIG. 4

give up its solids, and it is also kept fresh and uninoculated by the decomposing sludge below. Particles of sludge carried up by the rising gases do not disturb the settling process, nor are they caught by the current and carried out of the tank. They are allowed to undergo further decomposition in the gas chamber *d*.

The slope of the bottom of the settling chamber is usually not less than 45 degrees, and should be more nearly 60 degrees,

so that solids settling out will slide at once through the slot into the digestion chamber. The slot is made from 6 to 8 inches wide, to allow free passage for the larger particles, and the overlap to prevent rising gas or sludge particles from entering the settling chamber is usually at least 4 inches.

66. Depth of Tanks.—In addition to the feature of separation of sludge from liquid, the Imhoff tank differs from the septic and sedimentation tanks because of its greater depth. As originally built in Emscher, Germany, where this type of tank was first evolved by Dr. Karl Imhoff, the tanks were 30, or more, feet deep. Many tanks in the United States are 25 to 35 feet in depth, but there have also been several installations where the depth is not greater than about 20 feet. It has been assumed that the greater the depth of the tank, the better and easier to handle would be the sludge produced. Among the reasons given is the greater depth and the corresponding greater pressure under which the decomposition of the sludge goes on. Under this pressure, much of the gas produced by the decomposition is entrained and held in the sludge. When the sludge is drawn from the tank and discharged onto a bed for drying, these gas bubbles hold up the solids, allowing the water to drain out more rapidly.

67. Design of Tanks.—The sludge chambers of German tanks were designed on a basis of sludge deposit of 0.007 cubic foot per person per day for combined sewers, and 0.0035 cubic foot per person per day for separate sewers. American conditions differ, however, and tanks designed for use in the United States depart materially from German practice. In Germany the sewage flow per person is much less, and the sewage has greater concentration than in the United States. Folwell recommends about 14 cubic feet of sludge capacity per thousand people per day, with sludge chambers designed for a period of 6 months between cleanings. Metcalf and Eddy suggest providing storage space for 2.48 cubic yards of sludge for every million gallons of sewage; based on a storage period of 7 months, the volume of sludge to be provided for would be 528 cubic yards for a daily flow of 1,000,000 gallons of water.

The capacity of the settling chamber of an Imhoff tank is determined by the same general conditions as already outlined for septic and sedimentation tanks, that is, by the volume of sewage and by the detention period necessary for best results. When the sewage is septic or stale, a shorter detention period should be provided than in the case of fresh sewage. Ordinarily, the detention period will not exceed 3 or 4 hours.

The area and capacity of the gas vents, or gas chambers, are also important. Formerly it was the custom to make these 10 per cent. of the corresponding values in the sludge chamber, or slightly more, but the results were found to be too small for American practice. It is now the custom to make the horizontal area of the gas vents from 30 to 50 per cent. of the sludge-chamber area.

68. Advantages of Imhoff Tanks.—The two most important advantages of the two-story or Imhoff tank are a comparatively fresh effluent and a sludge less offensive and much more easily dried than that produced by the other methods of treatment. The inoffensiveness and the easy handling and quick-drying qualities of the sludge were, perhaps, the main reasons for the general use and acceptance of the Imhoff principle, but it has been found in practice that the advantages of a fresh effluent are, perhaps, even more valuable, especially in those places where the distance of flow through the sewers is long, and the sewage reaches the plant already stale or septic. In such places, any considerable additional loss of oxygen from the sewage would greatly increase the liability to odors, and the difficulty of secondary treatment.

Since 1915, there have been few large installations in the United States in which the Imhoff tank has not figured. It has its defects in operation, however, and while it is still a favorite of American engineers for the treatment of sewage, it is not regarded as a cure-all by which the difficulties of sewage treatment may be remedied.

ACTIVATED SLUDGE

69. Description of Method.—In the activated sludge method of sewage treatment, sludge is produced by the continuous agitation of sewage, for a considerable period, with an ample supply of oxygen present. This sludge is added to the inflowing sewage in proper proportion, generally to the extent of 15 to 25 per cent. of the volume of sewage; and agitation with the addition of sufficient air to provide for the oxygen demand then results in the removal by coagulation of a large part of the suspended and colloidal matter in the sewage. This coagulated material is deposited as sludge by sedimentation, and a part of it is used for the activation of the incoming sewage.

70. Sludge Tanks.—The agitation and aeration of the sewage is accomplished in continuous-flow tanks, in the bottoms of which are set air diffusers for the purpose of applying air continuously and uniformly to the sewage as it passes through the tanks. Tanks for activated sludge treatment are usually, though not always, rectangular in shape, and may vary in depth from 6 to 15 feet. The tank bottoms are of the sawtooth, the ridge-and-valley, or the inclined baffle type. In the valleys are placed the air diffusers, while the sloping ridges between the valleys deflect the settling solids onto the diffusers, so that they are further agitated.

71. Air Supply and Diffusion.—The air is usually supplied to the diffusers at a low pressure through a main extending under the center of the tank. From this main, branches, each of which supplies about 8 or 10 diffusers, lead off on both sides. These pipes should be of non-rusting material, so that the diffusers will not become clogged. Various materials have been used for diffuser plates, requirements for which, in addition to freedom from clogging, are small resistance to the passage of air, and ability to deliver air in a finely divided state, as bubbles over $\frac{1}{8}$ inch in size are not very effective. Iron pipes drilled with small holes have been used for diffusers but are not satisfactory, for they rust and clog; non-rusting metal

cloth operates well while new, but clogs under continuous operating conditions; wooden plates deteriorate rapidly and give rather unsatisfactory results. Perhaps the most satisfactory type of diffuser is the *filtros plate*, which is made of ground quartz sand. Filtros plates are usually set in Portland cement mortar, as iron holders frequently cause trouble from rust.

72. Operation.—Weather conditions have a marked effect on treatment by activated sludge, both very high and very low temperatures being detrimental. For ordinary sewage, a period of contact with the activated sludge of 4 to 6 hours may be necessary; with weaker sewage the time required will be less, and with strong sewage as much as 10 hours may be needed. From 1 to 2 cubic feet of free air is required per gallon of ordinary sewage, and considerably more for sewage containing trade wastes difficult to treat.

73. Results of Treatment by Activated Sludge.—Exceedingly good results have been attained by activated sludge treatment at the plants at Chicago, Illinois; Indianapolis, Indiana; and Milwaukee, Wisconsin. The bacterial removal averages well over 90 per cent.; the reduction of suspended solids in the sewage is much higher than that usually accomplished by other methods of treatment; and the effluent resulting from the process is stable, and needs no further treatment.

A great deal of sludge is formed by the process; the average yield is from 10,000 to 15,000 gallons of 98-per-cent. sludge per million gallons of sewage treated. The treatment of this sludge is difficult, for it cannot be dried in large quantities without being filtered under pressure. It makes a good fertilizer, however, and Milwaukee is drying sludge at a relatively great expense to make it available for this purpose.

74. Advantages and Disadvantages.—The activated sludge process is one of marked promise, but the involved and complicated processes of operation are a serious handicap, for the general experience in American cities has been that management necessary for efficient operation is not available. These

conditions, as well as high cost, generally limit the use of this process to cities of large size, though several successful small plants have been built. As yet, however, treatment by this method is but little beyond its infancy, and radical changes and improvements tending to simplification and a reduction in cost may be expected.

CHEMICAL TREATMENT

75. General Explanations.—When it is desired to remove from the sewage a large part of the suspended and colloidal matters, chemical precipitation may be employed as the treatment process. This treatment differs from simple sedimentation in that there are added to the sewage certain chemicals by whose agency a precipitate is formed that settles rapidly and carries down with it much of the undesirable solid matter. Under favorable conditions, good results can be obtained, and a clear, colorless effluent containing little suspended matter is produced. The cost, however, may be high, and the effluent is, in most cases, not stable, and may be expected to decompose with odor.

76. Requirements of Chemicals.—A precipitating agent should possess the following requisites: (1) It should be cheap and abundant; (2) it should cause rapid subsidence, or settling, of the precipitate formed; (3) it should not be poisonous nor produce poisonous compounds; (4) it should not color the effluent; (5) the precipitated sludge should part with its moisture readily; (6) since the cost of treating sludge is an important factor, the process of precipitation should not increase the quantity of sludge beyond what is necessary to effect the required degree of purification. There are few, if any, substances that meet all, or even many, of these requirements. The substances generally used as precipitating agents are lime, and the salts of aluminum and iron.

77. Arrangement of Precipitation Plants.—The method of treatment in which chemical precipitation is used is generally as follows: First, the precipitating agent is dissolved in water in a mixing tank, and then the solution is discharged

into the sewage in a small stream. The sewage is tested at frequent intervals by chemical reagents, and the amount of chemical admitted is proportioned to the amount of sewage and its content of organic matter, so that there will be no waste of chemical. It is important that the chemical be mixed thoroughly with the sewage, and, in order to accomplish this, the sewage, after the chemical is applied, is passed over cascades or through channels, called *fish ladders*, in which are set baffle walls that, by agitation of the sewage flowing through the channel, thoroughly mix the precipitating agent with the sewage. In a few plants, mechanical mixers are used, which agitate the sewage so that the chemical is evenly mixed throughout the volume. The mixture of sewage and chemical is then allowed to flow into settling tanks, which may be operated either continuously or intermittently.

78. Detention Period.—The period during which the sewage is held in the settling tanks depends on the composition of the sewage, and the kind and amount of chemical added. For continuously operated tanks, the detention period will be about 7 or 8 hours. Where the tanks are operated intermittently, British experience indicates that a holding period of 2 hours will yield satisfactory results.

79. Sludge Disposal.—The proper and economical disposition of the sludge formed by the chemical precipitation process is one of the serious difficulties connected with this method of sewage treatment. The volume of sludge produced averages four or five thousand gallons per million gallons of sewage, and the water content of the sludge varies from 90 to 95 per cent. Moreover, the sludge produced by this method does not yield readily to ordinary dewatering methods. Pressing is commonly used; when pressed, the sludge may be used for filling low lands, or to lighten heavy farm lands, but the fertilizer content is small. In many instances the sludge is taken out to sea and dumped. Drying on beds has been tried, but is not very satisfactory, owing to the slimy, greasy condition of the sludge.

80. Effect on Organic Matter and Bacteria.—While chemical precipitation removes a considerable part of the dissolved

organic matter in the sewage, as well as most of the suspended matter, the effluent, or partially purified sewage, resulting from this treatment is ordinarily putrescible and will decompose with odor. Furthermore, although a large percentage of bacteria is removed by the process of sedimentation, so many remain that the effluent must be considered to be potentially dangerous.

81. Cost.—There are few American cities now using chemical precipitation in treating their sewage. Satisfactory results are possible with this method of treatment, but usually the amount of chemical required for best results is such as to make the cost almost prohibitive. The process of handling and disposing of the sludge produced by this method is also costly, owing largely to the great volume, which is many times that produced by Imhoff or septic-tank treatment.

82. Treating Manufacturing Wastes.—Chemical treatment is particularly valuable in, and well adapted to, the treatment of manufacturing wastes, which are often the result of chemical processes, and in the treatment of sewage that is in part made up of such wastes and is less susceptible than ordinary sewage to the biological processes of treatment.

Among the industries which contribute wastes that are likely to be troublesome in the treatment of sewage are the tanning, wool-scouring, wool-washing, cloth-washing, dyeing-and-bleaching, wire-drawing, galvanizing, pulp-and-paper-making, brewing, and gas-making industries. Tannery wastes may contain lime and other chemicals. They usually contain much suspended matter, which should be removed by sedimentation and tank treatment before discharge into sewers. The chief difficulty in treating paper-mill and pulp-mill wastes is due to the particles of fiber or pulp, which are largely cellulose, and are unaffected by ordinary bacterial methods of treatment. When these wastes are sufficiently diluted with household sewage, such treatment is possible; but it is generally preferable, when the volume is large, to employ chemical treatment, sedimentation, or filtration, and afterwards to discharge the wastes directly into a stream. Grease from wool-scouring and wool-washing plants may interfere seriously with sewage treatment

if the quantity is great enough. Before such matter is discharged into the sewer, the grease should be removed by adding acid, and the wastes, if strongly acid, should be neutralized. Wastes from wire mills and galvanizing plants often contain much acid and may interfere with biological action. Sometimes it is possible to combine the iron contained in them with other chemicals, and to use the compounds thus formed to coagulate and precipitate the solids in the sewage.

Chemical precipitation may serve as a preliminary or an intermediate process in the treatment of manufacturing wastes before they are discharged into sewers, provided the chemicals employed do not destroy bacterial life and prevent biological action.

SLUDGE TREATMENT

83. Characteristics of Sludge.—Any form of tank treatment yields sludge, the amount and character of which depends on the method of treatment. The sludge from plain sedimentation tanks is very offensive; that from septic tanks is black and somewhat odorous; while that from Imhoff tanks is usually well digested and has no objectionable odor. Chemical precipitation produces a large volume of rather slimy and gelatinous sludge. Activated sludge processes yield a brown, flocculent sludge.

84. Removal of Sludge.—Sludge may be removed from tanks either by drawing off the supernatant liquid and then removing the sludge through drains, or by drawing off the sludge from the tank bottom through pipes without disturbing the sewage in the tank.

Where the former method is to be used, the tank bottoms should be constructed with a slight slope, so that the sludge deposit can be worked or washed toward the drains. In large tanks, the distances may be so great as to make this difficult, in which case several sumps or drains may be provided, with a section of the bottom sloping toward each.

Sludge removal without interfering with the action of the tank is usually accomplished by making the slopes to the drains much steeper, preferably 45 to 60 degrees, so that when the

drain valves are opened, the weight of the liquid above will force the sludge through the drains. By this means, the sludge may be raised easily to within a short distance of the top of the liquid in the tank.

Mechanical appliances are also used to aid in the removal of the sludge. One device consists of arms or bands mounted on a revolving carrier. The sludge is thus concentrated at the center of the tank, whence it can be discharged by gravity or pumped out of the tank. In another arrangement, flights suspended between two chains scrape the sludge into a hopper at one end of the tank.

85. Methods of Treating Sludge.—The process of sludge disposal consists of two parts; namely, removing the water and disposing of the dry solids. Perhaps the most used method of removing the water is to spread the sludge on specially constructed, porous beds, where it is allowed to dry. Sludge may be dewatered mechanically in sludge presses. It is difficult to dewater the sludge from plain sedimentation tanks by either method unless it is digested first in a separate digestion tank. Sludge from septic tanks, Imhoff tanks, and separate digestion tanks will dry on beds, the drying of the sludge from the latter two classes of tanks being much more rapid. The sludge from chemical precipitation may be dewatered mechanically but dries slowly on beds. If the sediment from activated sludge treatment is to be dried on beds, it must be spread out in a very thin layer; since a very large area of beds is required, lagoons or trenches are sometimes used into which the sludge is run and in which it is allowed to stand until evaporation and seepage accomplish the work of drying out the sludge. This process is generally odorous and unsatisfactory.

Sometimes, the dried sludge is used for fertilizer, but in most places, except where the activated sludge process of treatment is used, it is considered to have such little value for this purpose that its disposal is a difficult problem. Some cities, which are situated along the sea coast, haul the sludge out to sea in specially constructed scows or ships and dump it many miles from shore.

86. Sludge-Drying Beds.—Sludge-drying beds are built on the general plan of a sand filter. The usual construction is a shallow layer of sand on a course of 6 to 8 inches of graded gravel, the whole underdrained with lines of open-joint tile. At most plants the area of sludge bed allowed for each person contributing sewage to the plant is 1 or 2 square feet, but according to Metcalf and Eddy, this area may vary with the type of treatment, the rainfall, and the climate from ½ to 7 square feet per person. Distribution of the sludge is accomplished by a tile or iron pipe, which is elevated about a foot above the surface of the bed, and which discharges onto a concrete apron. The time required for drying a deposit of sludge from 8 to 12 inches thick will vary with the weather from about 2 weeks to 2 months.

87. Mechanical Dewatering.—Sludge pressing, whereby the sludge is forced under considerable pressure into chambers lined with cloth, generally heavy duck, has been in use for many years. Pressing is a rather disagreeable process, and some sludges do not give up their water readily without the admixture of some chemical, usually lime. This is added to the extent of 20 to 100 pounds per 1,000 gallons of sludge. Pressing usually reduces the moisture content of the sludge to about 70 per cent. Centrifugal dewatering machines are also used, but have not given such good results. In the most successful of these, there is a drum that is subdivided into pockets by vanes. The sludge is driven into the pockets by centrifugal force, whereas the water remains in the center.

88. Sludge as Fertilizer.—In general, the value of sewage sludge as a fertilizer has been overestimated. While the richer sludges have as much as 3 per cent. of nitrogen, the nitrogen content is usually much less and only a small part of it appears to be available to most soils. Sludge also contains potash and phosphoric acid, both valuable fertilizing agents. However, some time is required after the application of the sludge before the fertilizing elements in it become available. Therefore, though sludge undoubtedly possesses some fertilizing value, its use as a fertilizer in the United States has been

rather discouraging. Before it has much value, the water content must be reduced to about 10 per cent. and it is often necessary to remove the contained grease, which is detrimental to the soil if present in any considerable quantity. All these operations are costly, and it is difficult to sell sewage fertilizer in competition with commercial products. The activated sludge method of treatment yields the most valuable sludge from a fertilizing standpoint.

SECONDARY TREATMENT OF SEWAGE

INTRODUCTION

89. Aim of Secondary Treatment.—Primary treatment removes a large part of the organic matter in sewage, but the resulting effluent is, in reality, merely a less objectionable form of sewage. The coarser solids have been removed and the first stage of decomposition has set in. All, or nearly all, of the oxygen has been used up in the partial purification that has already taken place and more must be supplied to prevent objectionable decomposition. Secondary treatment must oxidize the remaining organic matter to stable forms, so that decomposition with odor will not take place; it must also reduce the number of bacteria.

90. Methods of Secondary Treatment.—As already mentioned, common methods of secondary treatment include sand filters, contact beds, and trickling filters; other methods are irrigation and disinfection. Auxiliary settling tanks are sometimes used in addition to one of the forms of secondary treatment already mentioned, generally trickling filters. The choice of the method to be adopted will depend on the size of the plant, the local topographic conditions, the degree of treatment necessary or desirable, and the form of primary treatment.

The various methods of secondary treatment will now be considered in detail, and their advantages, disadvantages, and limitations outlined.

INTERMITTENT SAND FILTERS

91. Principles Involved.—Intermittent sand filtration is treatment of sewage by applying it at intervals to beds of sand which act as filters. By this process of treatment, purification is effected in two ways: first, by the bacterial action in the interstices of the bed as the sewage passes downwards; and second, by the mechanical straining action of the sand. The oxidation and stabilization of the putrescible portions of the sewage is carried on by aerobic bacteria. These bacteria work best in the presence of oxygen, and, therefore, it is necessary to apply the sewage to the bed of sand intermittently, so that air is taken into the interstices of the bed between doses of sewage. It is also usually necessary to underdrain the bed to insure quick removal of the liquids, and to allow access of air to the interior. In some places untreated sewage is applied to sand beds, but in most cases the grosser solids are removed from the sewage by screening or sedimentation before the sewage is applied to the beds.

Under favorable conditions of construction and operation, a sand filter yields an effluent that is practically pure, and not subject to decomposition.

92. Natural and Artificial Filters.—A sand filter consists essentially of a level bed of sand or gravel, a system of troughs or pipes for distributing the sewage over the surface, and an adequate underdrainage system. Filters may be classed under two heads, *natural* and *artificial*. When a bed of sand or gravel is available at the necessary location and elevation, a sand filter is obtained simply by removing the top soil and grading the bed. There are few sections of the United States, however, where the soil conditions are such that natural filters may be utilized. Artificial filters may be constructed anywhere. The operation and general construction are the same for both kinds of filters.

93. Construction of Filter Beds.—The filter material is preferably a layer of clean, sharp sand having an effective size of .3 to .5 millimeter. Uniformity of the sand is advantageous.

The depth of the layer varies from 30 to 42 inches. Underlying the sand is a bed of gravel to assist in underdrainage. Clay and loamy soils are not suitable for filters. If the sand is composed of grains of different sizes, the finer sand should be on top; otherwise clogging will occur within the bed where it cannot be remedied easily.

94. Underdrainage.—In some natural filters underdrainage will not be necessary, but in most places it is desirable. Artificial filters must be underdrained. The upper 12 inches of the filter bed effects the most purification, and each succeeding foot is less effective. Nevertheless, the underdrains should be 4 or 5 feet below the surface. They are generally spaced 30 to 40 feet apart. Drainage is often accomplished by means of a center underdrain toward which the bottom of the bed slopes.

95. Size and Shape of Filter Units.—The area of filter beds required for adequate treatment of the sewage of a community depends on the amount of sewage produced and on the degree of preliminary treatment. Under average conditions, it is feasible to apply to each acre daily from 75,000 to 100,000 gallons of clarified sewage. This is roughly equivalent to the sewage produced by from 750 to 1,000 people.

The shape and size of the units will be determined largely by topographic conditions, but the operation must be considered also. The beds should not be so large that the plant will be crippled by taking one out of service, and the shape should be such that equal and uniform distribution of the sewage over the surface is possible.

96. Dosing Beds.—As the term intermittent sand filtration indicates, the sewage is applied to the filter beds at intervals. Since the surface of a bed usually is practically level, the sewage may be distributed by means of wooden troughs, by pipes, or simply from an outlet. The flow is best regulated by collecting the sewage in a tank and discharging it at intervals by means of a siphon. In order to reduce the amount of attention required, two or more siphons may be set in the same

chamber, and so arranged as to discharge sewage alternately upon different beds. In each dose, the beds are covered to a depth of 2 or 3 inches. The period between discharges will vary with the requirements of the operating conditions; when

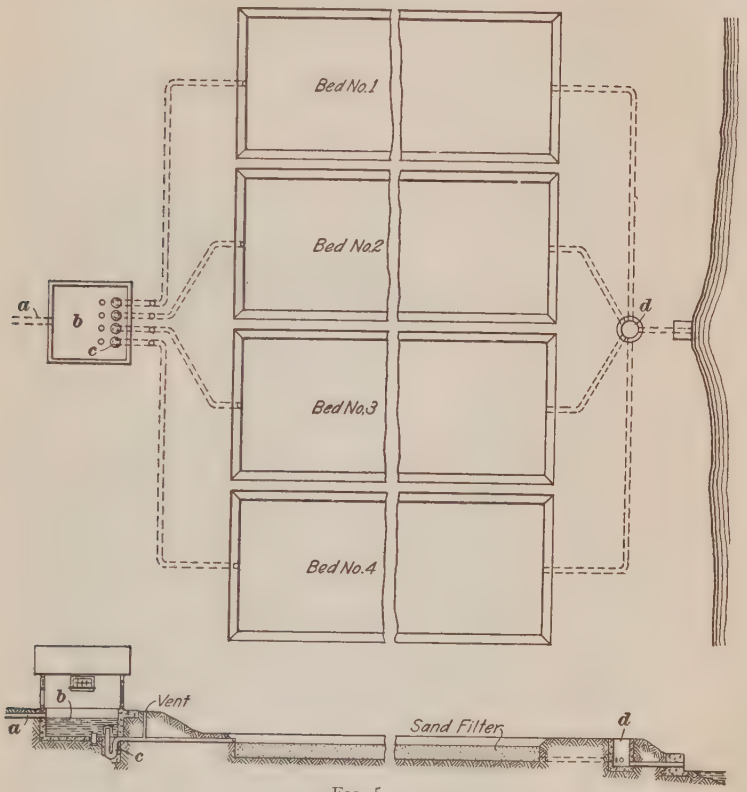


FIG. 5

the dose is 3 inches deep, a bed having a daily capacity of 85,000 gallons per acre will normally be dosed once every day.

97. Operation.—In Fig. 5 is shown a plan and section of a typical plant layout with four beds, the distribution system and divisions between the beds being illustrated in Fig. 6. The sewage flows from a settling tank through the inlet *a*, Fig. 5, into the dosing tank *b*, which contains the siphons *c* for con-

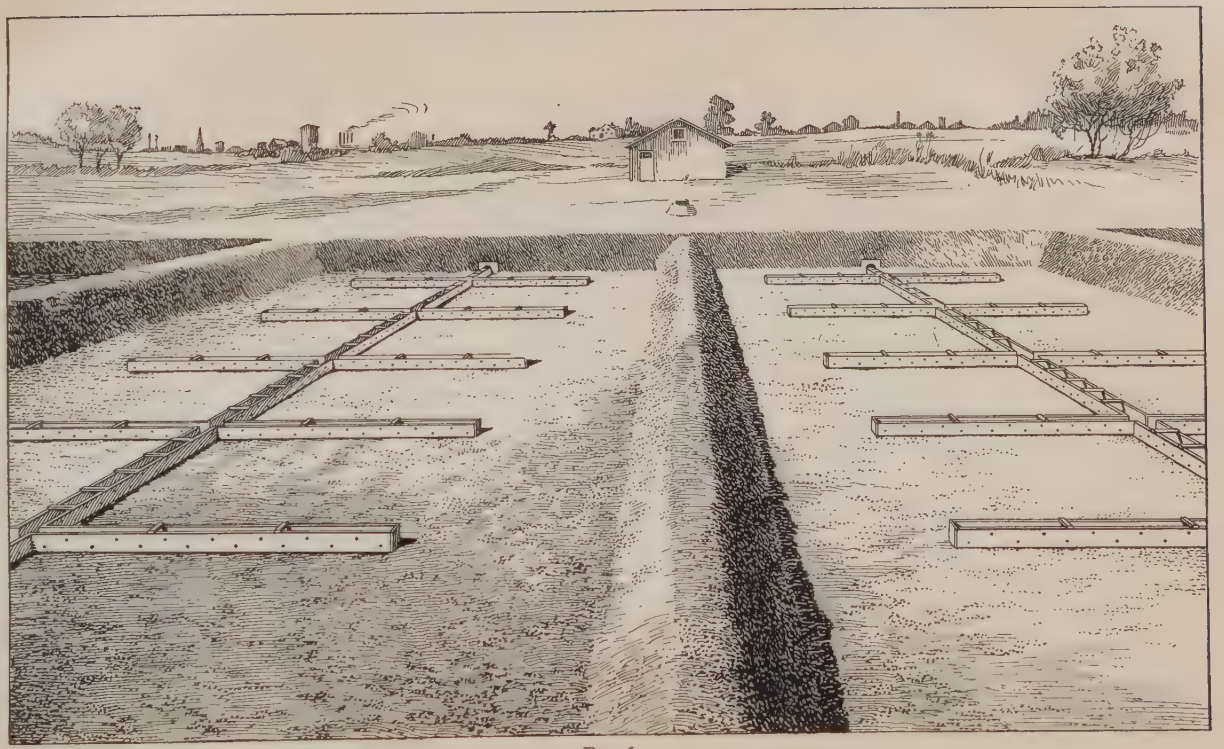


FIG. 6

trolling the discharge to the beds. The effluent from all the beds is carried to the outlet *d*, whence it is discharged into the stream.

98. Supervision.—The beds of intermittent filters require considerable attention. In order that the nitrifying bacteria in any one bed should not be overworked, the flow of sewage must be diverted from one bed to another at frequent intervals. Less frequently, the tops of the beds must be raked to break up the material that collects in the uppermost layer of sand. It is usual to provide an excess number of beds so that one or more can be thrown out of service for cleaning or resting. When raking no longer prevents the accumulation of puddles on top of the bed, about an inch of sand should be removed from the top. It is not good practice to plow or rake the bed to a depth greater than an inch or so, for fine sand or organic matter may be deposited below the top layer, where it will collect fine material and clog the bed. After the bed has been scraped several times, the sand lost in this way should be replaced. Weeds and other vegetation should not be allowed to grow on the bed.

In northern climates, special precautions must be taken during the winter to prevent freezing of the sewage. It is the custom to ridge or furrow the beds to a depth of 6 to 12 inches. The bed is then dosed to the tops of the furrows. Ice forms on the surface and protects the bed, which can then be dosed in the usual way, the sewage flowing through the furrows underneath the ice.

99. Results of Treatment.—When intermittent sand filters are operated properly, the suspended material in the sewage is strained out in the passage through the sand, and the organic matter remaining in the interstices of the sand bed is transformed to stable compounds. Therefore, the effluent is practically pure, clear, odorless, and not putrescible. Nevertheless, it is still potentially dangerous because of the bacteria it may contain. The greatest disadvantage of intermittent sand filtration is the comparatively low rate of operation, which

makes the cost of construction of artificial filters for a city of medium size almost prohibitive. Even with a capacity of 1,000 people per acre, a city of 100,000 population requires 100 acres, the acquisition of which would be difficult and costly. Comparatively few large cities in the United States are situated within reach of natural sand or gravel deposits suitable for use as sand filters.

100. Rapid Sand Filters.—Attempts have been made to filter sewage in much the same manner as water and at approximately the same rate. The attempts have not been successful, for the filters appear to store up the excess organic matter until clogging occurs.

CONTACT BEDS

101. General Features.—It was early recognized that the mechanical straining out of the suspended matter in the sewage while passing through a sand filter was the least important part of the process of purification, and that the actual work of nitrification was accomplished by the passage of the sewage in very thin sheets over the surface of the sand grains. It was also recognized that the small grains of sand prevented the application of the sewage at rates greater than 100,000 gallons per acre per day. The solution appeared to lie in the use of filters of coarser grains, such as gravel or broken stone. These, however, are so porous that sewage applied to the surface in the same way as to a sand bed passes directly through them without treatment. Therefore, the fill-and-draw method, with the filter material contained in a tight tank, was employed. It has been found necessary to hold the sewage in contact with the filter material for some time before the bed is emptied and the bacteria are allowed to work. Consequently, this type of filter is designated as contact beds.

Contact beds work on the same general principle as do sand filters. In other words, a large part of the solid matter and some of the colloidal matter is retained in the interstices of the bed, while aerobic bacteria in the slimy film covering the contact particles convert the organic matter into stable substances.

102. Method of Operation.—The method of operation is the same on all contact beds, though the cycle of operation may vary considerably. The bed is filled with sewage, allowed to stand full for a time, emptied, and allowed to stand empty for a time. The time required for these operations will vary with local conditions. The times of filling and emptying have little influences on the quality of the effluent, but the periods of standing full and resting are very important.

The time that the bed is allowed to stand full must not be so short that bacteriological action cannot take place, nor so long that anaerobic decomposition will begin. The contact period varies, in different plants, from 30 minutes to 4 or 5 hours. Generally little benefit is obtained from contact periods exceeding 2 to 3 hours. Fairly good results are often obtained with contact periods of 30 to 45 minutes, but longer periods should be employed where possible.

It is during the time that a bed stands empty that the bacteria accomplish the work of reducing the organic matter deposited in the contact material. Consequently, this period is important. A 4-hour resting period is considered enough for average conditions. It may be necessary to shorten this period, but best results will seldom be attained.

103. Contact Material.—Experience has shown that contact material should be durable, large, and sharp. Good, hard trap rock or sandstone makes excellent contact material. Crushed blast-furnace slag may be used; but in some places it is not considered to have the required durability, for some slag is friable and will decompose in a few years under the conditions of use in a contact bed. For best results the stone should be from $2\frac{1}{2}$ to 4 inches in size. Formerly smaller stone was used, but the larger pieces have greater capacity, do not clog so easily, and yield practically as good an effluent.

104. Design of Contact Beds.—Contact beds may be arranged in the same way as filter beds are. The size of contact beds is quite important. They should not be so large that considerable time is required to fill them; there should be a sufficient number to give a certain degree of flexibility, so

that a bed may be cut out of service for occasional resting periods without overloading the others. It is desirable to have at least three or four beds for any plant.

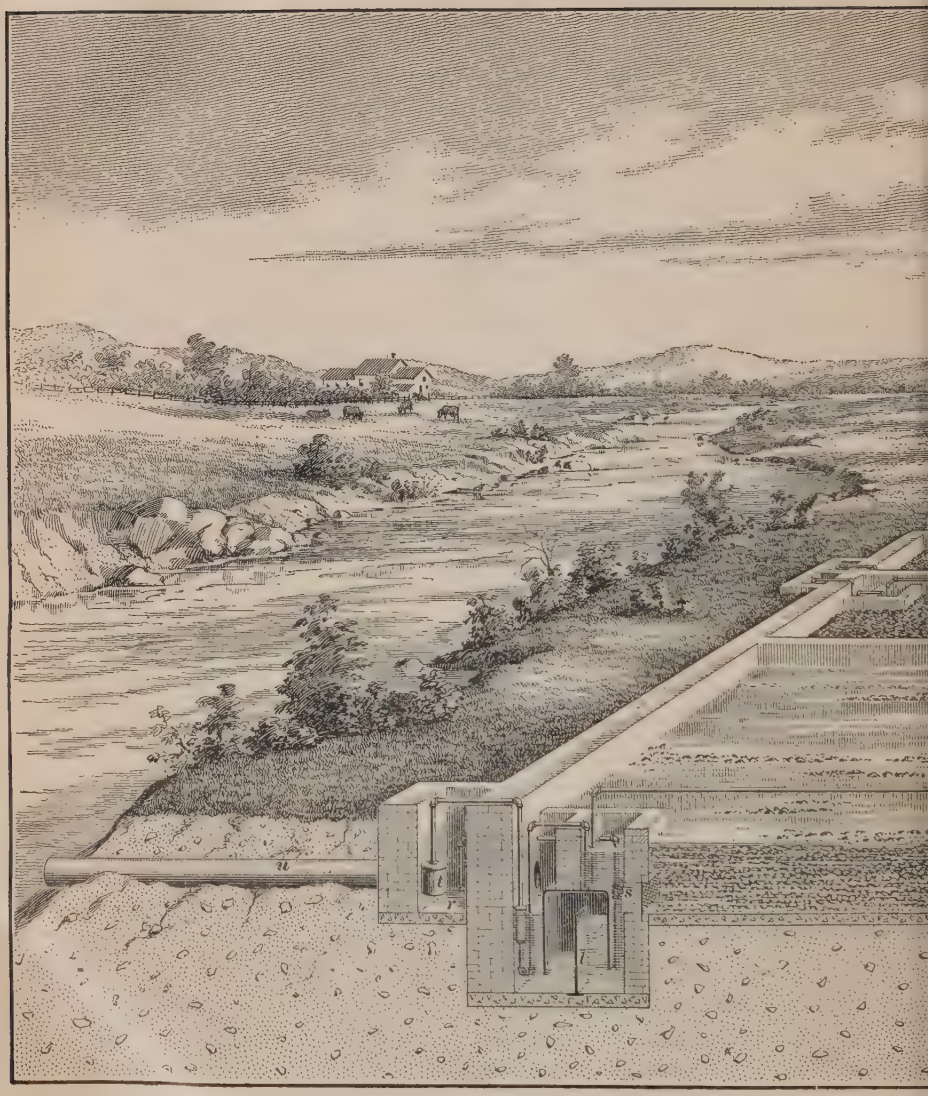
Beds should not be too deep. The first beds installed were made only about 30 inches deep, but present-day practice is to make the beds about 4 or 5 feet deep. A greater depth than 6 feet is not generally desirable. A requirement of the drainage system is that it empty the bed quickly and completely. A drain system of half tile may be used, or the floor of the bed may be constructed to act as a drain.

105. Rate of Operation.—The rate of applying sewage to contact bed depends, of course, on the depth of the bed. Although in many places, owing to overloading, lack of intelligent operation, etc., rates as high as 1,000,000 gallons per acre per day are maintained with beds 5 feet deep, the best rate of operation for contact beds is about 500,000 to 600,000 gallons per acre per day, that is, 100,000 to 125,000 gallons per acre per day per foot of depth. Operation at these rates gives satisfactory results, whereas operation at the higher rates does not, as a rule.

It will be seen that contact beds, though not producing such a good effluent, will handle sewage at rates 6 or 8 times as high as those used with sand beds. This is an important advantage under most conditions.

106. Control of Operation.—The times of filling contact beds, letting them stand full, emptying them, and allowing them to rest can be controlled either by hand or automatically. Because of continual variations in the strength and volume of the sewage and in the condition and capabilities of the individual beds, manual control may be better in larger plants where more intelligent labor and more skilled supervision are available. In smaller plants, on the other hand, automatic control usually is found to be better and cheaper.

Where manual control is used, gates and piping must be provided so that the flow of sewage can be diverted to or from any beds, and the beds can be drained. For automatic control,



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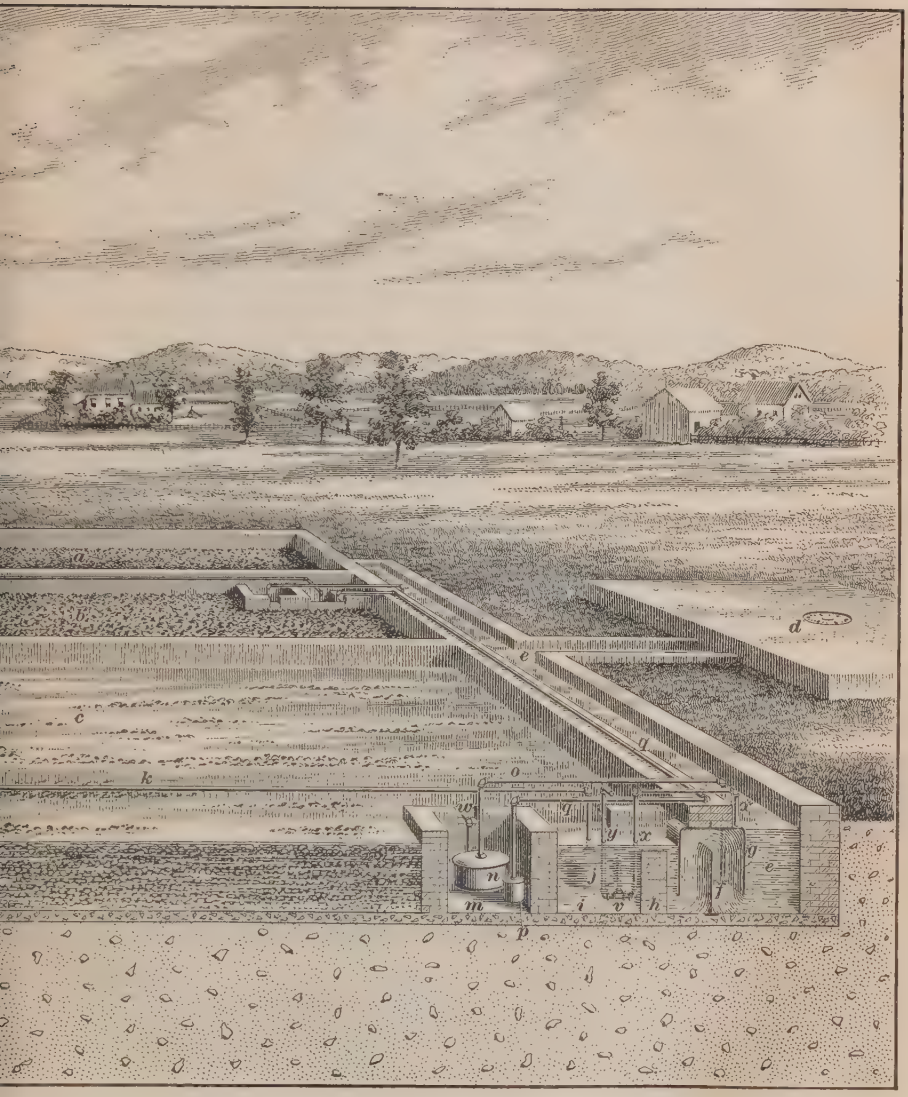
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one set of apparatus is required to fill the beds and another to empty them. Air-lock feeds are customarily used for filling, and timed siphons for emptying.

Equipment for operating contact beds automatically is shown in Fig. 7. The beds *a* and *b* are assumed to be empty and the bed *c* is being filled. Another bed, not shown, is full. Sewage flows by gravity from the dosing tank *d* through the inlet sluice *e* and over the weir *f* inside of the bell *g*. It then passes over the weir *h* into the inlet locking chamber *i*, and compresses the air in the bell *j*, shown dotted. As the sewage rises, the increase in pressure in *j* is communicated through the pipe *k* to another bell covering the weir *l*, over which the effluent from the bed must flow before it can leave the bed. While the bed is being filled, the air pressure in the bell keeps the level of the effluent depressed and prevents the discharge of the effluent.

When the bed is full, the sewage overflows into the locking chamber *m* and displaces air in the bell *n*. This air passes through the pipe *o* to the top of the bell *g* and prevents the sewage from flowing over the weir *f*. At the same time, the air in the small bell *p* is displaced and, passing through the pipe *q*, starts the flow of sewage into the bed *b*.

The sewage remains in the bed *c* until the timing chamber *r* has been filled through the timing cock *s*, shown dotted, which is set to give the desired time of contact. As this chamber *r* fills, the air in the bell *t* is compressed and the level of the sewage to the left of the weir *l* is lowered until it falls below the bottom of the bell covering this weir. The air then rushes out of the bell, and the sewage starts to flow over the weir *l* and is discharged through the outlet pipe *u*.

After the other beds have been filled in turn, the pressure in the small bell in the inlet locking chamber of the last bed filled blows the seal of the release trap *v*. The air pressure in the bell *g* is thus relieved and sewage is again admitted to the bed *c*. In order to supply air to the bell *n*, the chamber *m* is emptied through the siphon *w* every time the liquid is drained from the contact bed. The pipe *x* leads from the release trap *v* to the top of the bell *g* and, when the trap is blown, provides an out-

let for the air held in the bell. The pipe *y* is connected to the bell in the inlet locking chamber of the bed that is not shown.

107. Clogging of Filter Stone.—Every filling of a contact bed leaves behind a deposit of organic and mineral matter, not all of which is reduced or flushed out. Consequently, there is a steady, though slow, filling of the voids in the bed. The time required for the beds to fill to a degree sufficient to reduce materially the capacity of the bed varies with the amount and strength of sewage, and the suspended matter in it. New beds have voids of 40 to 50 per cent. After some use, the voids will decrease to about 30 per cent., near which amount they may remain for quite a long period. When the voids no longer exceed 20 per cent., the bed should be washed or the contact material replaced with new. Washing may be expensive, per yard of filter material, in small plants.

108. Washing Filter Material.—A good way to wash filter material is to pass it in contact with a considerable volume of water through an inclined cylinder, the lower part of which is perforated. A cylinder of the type used for screening stone, placed at about the angle used for that work, is suitable. It should be rotated at such a speed that a point on the curved surface has a velocity of about 5 feet per second. The stone is washed in the upper part of the cylinder, and the water carrying the dirt escapes through the perforations. The stone, after washing, is as good as new, and the cost of washing the stone is usually less than that of replacing it.

109. Degree of Treatment.—The rate of treatment in contact beds is very much greater than in filter beds, but the effluent in the former case is not nearly so pure. It is generally slightly turbid and contains a small amount of suspended matter, which is still putrescible, although considerable oxidation has taken place. For the great majority of cases, contact-bed treatment, combined with dilution, is sufficient to prevent nuisance.

Contact beds are often built in series. In this arrangement the sewage, which is generally the effluent from a septic tank,

is treated in the first bed, and the effluent from that bed passes to a second bed; sometimes the effluent from this second bed is treated in a third. A progressive purification is obtained in the passage of the sewage through the beds. Experience has shown that the use of double beds is often an advantage, for a somewhat greater quantity of sewage can be treated than with single beds of the same area. A disadvantage is the head consumed; the loss of head in triple contact beds usually makes them impracticable.

TRICKLING FILTERS

110. General Description.—While satisfactory in many ways, contact beds are limited in capacity and require cleaning at intervals; it was to avoid these limitations that trickling filters were devised. Instead of being held in contact with the filter material in tight chambers or tanks, the sewage is applied intermittently to the surface of the filter material and trickles down through the bed, meanwhile undergoing oxidation.

As in the case of contact beds, the sewage must be applied so that the bacteria have an opportunity to work under aerobic conditions. The greatest difficulty is to obtain an even and regular application of the sewage over the entire surface of the bed. The sewage is usually distributed by means of spray nozzles, as shown in Fig. 8. The bottom of the bed is open, and the effluent escapes freely.

111. Preliminary Treatment.—It is necessary to remove as much as possible of the suspended matter in the sewage before it is applied to the filters in order to prevent clogging of the filters. Because of the degree of preliminary treatment desirable, sedimentation is generally used, since screens do not remove enough of the solids. The common practice at the present time is to use the Imhoff tank or the sedimentation tank with separate sludge digestion as a preliminary to filtration.

112. Filter Material.—The requirements for filter material are much the same for trickling filters as for contact beds, though the pieces of stone may be a little larger. The usual

FIG. 8

specifications call for stone that is 2½ to 4 inches in size, durable, and sharp.

113. Depth of Filter Material.—Purification is dependent more on the volume of filter stone than on any other factor, if proper dosing and primary treatment are provided. About the same volume of sewage can be treated on one acre of beds 9 feet deep as on two acres 4½ feet deep. Experience has shown that a somewhat greater depth is desirable than in contact filters, and present practice is toward trickling filters 8 or 9 feet in depth. The arrangement of the filters should be such that a portion of the bed can be cut out for resting or cleaning without disturbing the operation of the remainder. Often the filter is divided by walls into a number of beds of such sizes as local considerations may dictate.

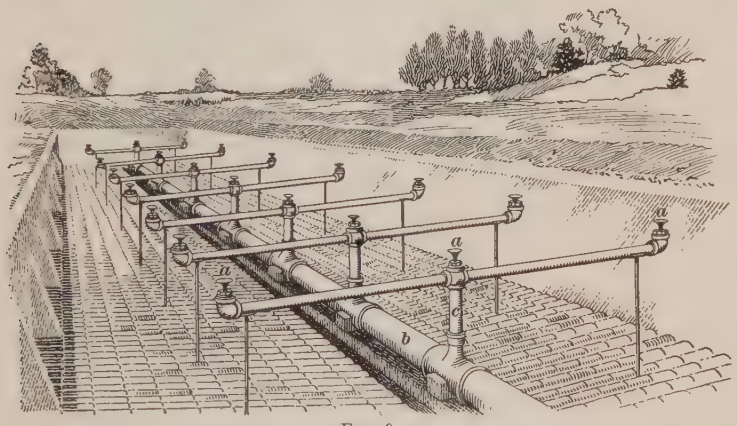

FIG. 9

114. Features of Design.—Units of sprinkling filters are usually, though not always, rectangular in shape, much depending on the local topographic conditions and on the type of distribution system. A minimum of three units should be used, and more are advantageous, to allow throwing a portion of the plant out of service for resting or repairs, without overloading the remainder.

The arrangement of the distributing system for a small plant is shown in Fig. 9. The sewage is delivered to the spray

nozzles *a* through laterals *b* running each way from the main pipe carrying the sewage to the filters. Laterals are usually of cast iron, and are laid from 1 to 3 feet beneath the surface of the bed; occasionally they are laid just above the filter floor or on top of the filter. Inasmuch as the head on the pipes is small, concrete pipe or vitrified pipe encased in concrete is sometimes used in the distributing system. The risers *c*, which carry the sewage from the laterals to the nozzles, are generally of cast iron, and extend to, or sometimes above, the surface of the bed.

Trickling filters must be underdrained; they are always built with a tight floor of masonry, so that an adequate underdrainage system is usually possible. In Fig. 9 is shown an underdrainage system of half tiles laid on a concrete floor, a favorite type of construction because of its cheapness and good qualities. Slotted or perforated slabs of concrete or vitrified tile are also used in quite a number of places. Often the grooves or channels in the floor under the slabs extend through the walls so as to allow flushing or cleaning out. Such an arrangement also aids in ventilation, which is essential. Additional ventilation facilities are sometimes obtained by leaving holes in the walls, or by making loose walls of stone or other material act as the retaining walls for the filter.

115. Dosing Beds.—The application of sewage to trickling filters is usually intermittent. For instance, the sewage may be applied for 3 to 5 minutes, and the application then discontinued for 3 to 10 minutes. Much the same equipment is used for controlling the flow of sewage, as in the case of sand filters. Dosing tanks collect the flow until a predetermined amount has been collected, when the accumulation is discharged rapidly through spray nozzles or over splash plates. In the former the sewage is sprayed into the air and falls back on the bed; in the latter, a small stream of sewage is allowed to fall onto a splash plate, which distributes the sewage over a section of the bed. Traveling distributors also have been used, but the general practice in the United States has been to use spray nozzles.

The dosing tanks are so constructed that, when full, there will be a head of 8 or 10 feet on the nozzles. When the tank becomes filled by the discharge from the sewer, a siphon discharges it. As the level in the tank falls, the head on the nozzles decreases until it reaches zero with the tank empty. By making the tank with sloping side walls, a longer discharge under the greater head is obtained. When the tank becomes empty, the siphon cuts off and stops the flow to the nozzles. One tank may be sufficient, but it is generally preferable to use two or more tanks.

116. Arrangement of Nozzles.—Spray nozzles may be circular or square. In Fig. 10 (*a*) is represented the usual

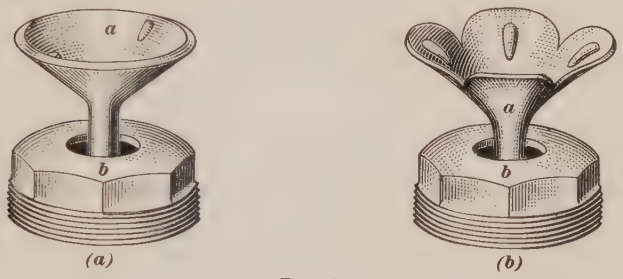

FIG. 10

type of circular spray nozzle, and in (*b*) is shown a square nozzle. The spindle *a* is either threaded or locked in the dome *b*. Fig. 11 shows the method of spacing the nozzles to obtain even distribution of the sewage over the surface of the bed. Without allowing any overlap on adjacent areas, circular nozzles placed at the centers of squares would cover only about 75 per cent. of the bed area. Even when placed at the apexes of equilateral triangles, as shown in Fig. 11 (*a*), they do not cover the beds very well. If a nozzle is inserted at each point *a* and the spray limits are represented by the full circles, there will be an area *b* at the center of each triangle which does not receive any sewage. Therefore, some overlapping is provided. For example, if nozzles are located at *a* and the spray limits are indicated by the dotted circles, there will be an area *c* in each triangle that receives sewage from all three of the adja-

cent nozzles. In Fig. 11 (*b*) is shown the spacing of square nozzles. The square around each nozzle *a* is indicated by the full lines and the spray limits are represented by the dotted lines. There is, therefore, considerable overlapping.

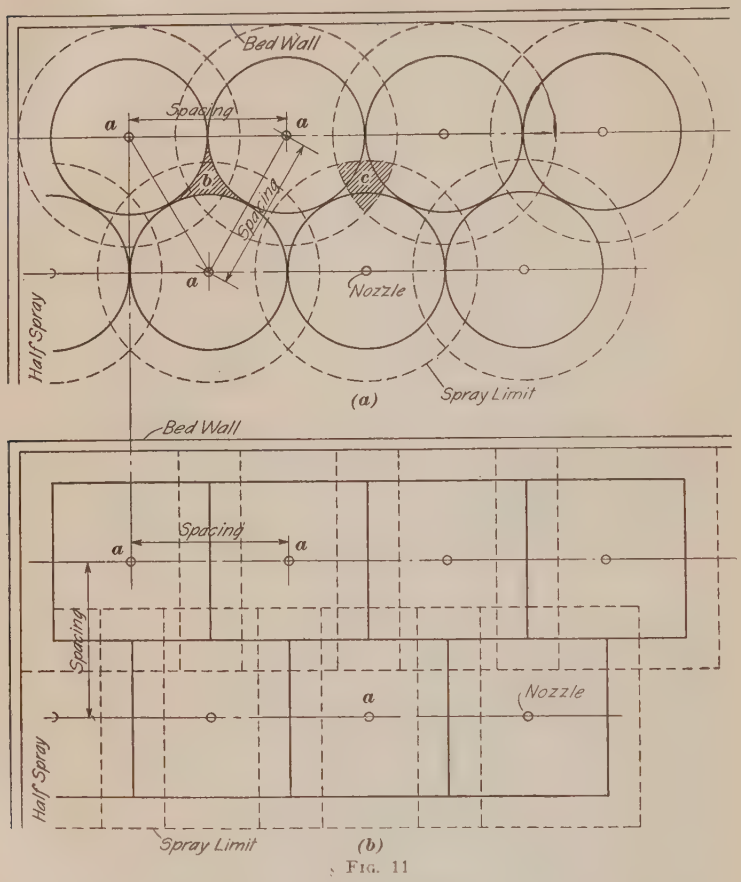

FIG. 11

117. Rate of Operation.—Trickling filters may be operated at even higher rates than contact beds and, therefore, require less area than the latter. Usual rates for beds 8 feet deep will vary from 1,500,000 to 2,000,000 gallons per acre per day, which is the equivalent of 200,000 to 250,000 gallons per acre per day per foot of depth.

118. Results of Treatment.—The effluent from a properly operated trickling filter is usually not putrescible, and, when discharged into a watercourse, rarely causes odor or nuisance. In the spring, when a portion of the solids stored during the winter are discharged, or *unloaded*, it may carry considerable suspended matter. This may be removed, if desired, by secondary settling tanks with a short period of sedimentation. The effluent, after sedimentation, is still somewhat turbid and colored, but generally quite stable. In well-operated plants the bacterial removal in trickling filters is about 80 to 90 per cent.

119. Advantages and Comparisons.—In most places trickling filters have superseded other forms of secondary treatment. Contact beds are used only in small installations or in those places where lack of available head prevents the use of trickling filters. Contact beds 4 feet in depth require a minimum of 5 feet of elevation for their operation; trickling filters 8 feet in depth will use up at least 11 feet of head.

Under some conditions, trickling filters may cause odors. For most places, however, they are the most economical and satisfactory form of secondary treatment now available.

IRRIGATION AND SUB-SURFACE DISPOSAL

120. Surface Irrigation.—Irrigation with sewage performs the double duty of furnishing water and fertilizing elements. Perhaps the most valuable element is the water itself. Although nitrogen, potash, and phosphoric acid, all valuable fertilizing agents, are found also in sewage, they are not generally in a form available for direct and immediate use by plant life. The greases, fats, and soaps tend to clog the soil and interfere with the process of irrigation.

The sewage is often clarified by screening or some other method of primary treatment before it is applied to the land. Careful management is necessary to prevent nuisance and the spread of disease. Generally the sewage is applied to the surface of a field by means of numerous ditches throughout the field. Sewage farms rarely, if ever, yield profits. In a few

cases, however, it is possible to obtain revenue from them that will cover a part of the expense of disposing of the sewage. While sewage farming is rather common in Europe, it is little used in the United States; where it is used, the sewage is valuable rather because of the water it contains than for any other reason.

Among the objections to the disposal of sewage by irrigation are: (1) the difficulty of harmonizing the demands of agriculture and the higher demand for proper and safe sewage disposal; (2) the difficulty of getting suitable land near the centers of population; (3) the local opposition to the establishment of a sewage farm in any locality; (4) the expense of pumping the sewage, or transferring it by other means, to a remote area.

121. Subsurface Disposal.—The effluent from sewage treatment plants may be disposed of through lines of open-joint tile laid 12 to 24 inches below the surface of the ground. The effluent soaks through the joints in the tile line and into the ground. There is no odor from such an installation, and the requirements of sanitary disposal are attained.

122. Limitations of Use.—The method of secondary treatment by subsurface disposal is limited to plants, such as for an institution or a single home, where the flow of sewage is small, because the construction of a subsurface system for the area needed to care for the sewage from a plant of considerable size would entail a heavy cost. There are the same disadvantages as are encountered with surface irrigation. When the ground is wet and unable to receive much sewage, the flow of sewage is liable to be a maximum; not all soils readily absorb the sewage; and clogging of the soil will result from the application of too much sewage.

123. Tile Systems.—The material most frequently used for subsurface lines is open-joint drain tile, without bells. This is laid on a bed of slag, gravel, broken stone, or other porous material, with the ends of the tile sections spaced about one-quarter of an inch apart to allow proper leaching. For the distributing lines, 4-inch tile is generally used; for the main

lines, larger pipes, the size depending on the flow of sewage, are better. For the larger plants, dual systems of tile are preferable, so that resting periods may be allowed at frequent intervals.

124. Construction of Tile Lines.—The length of a tile line for the proper disposal of the sewage depends directly on the absorptive capacity of the soil, and will, therefore, differ widely in different sections of the country. In clay soil, where the absorptive power is small, at least one foot of properly laid tile line is necessary for a gallon of sewage per day. In sandy soils, less tile is required. The layout of the line is not so important, as long as the parallel runs are at least 5 or 6 feet apart.

125. Method of Laying Tile.—A trench is dug 10 or 12 inches deeper than the desired level of the tile line, and filled with gravel, broken stone, cinders, or other porous material, to the desired level. On this bed, the tile is laid. A fall of 4 inches to 100 feet is desirable for most conditions; where the soil is exceptionally porous, a slightly greater fall may be used.

126. Filter Trench.—A variation of the preceding method, especially suitable for use in extremely tight soil, is the *filter trench*. The trench is about 4 feet deep, and in the bottom is an open-joint drain, or collecting pipe, having a free discharge into a ditch or stream. This drain is overlaid with about 3 feet of gravel, cinders, broken stone, or other porous material, on top of which the effluent from the septic tank is discharged. As the effluent trickles down through the stone somewhat in the manner of a trickling filter, considerable purification is effected.

127. Application of Sewage.—The usual method of distributing the sewage along the trench and on top of the filter material is by means of an open-joint tile line, similar to that previously described. It is preferable to control the discharge by a siphon. When the overflow from a small septic tank discharges directly and continuously into the tile line, that part of the line nearest the tank receives more than its share of the sewage, and soon becomes clogged. When a siphon is

installed, it collects the flow during a considerable period and then discharges it rapidly, the sewage being distributed equally to all parts of the system.

DISINFECTION

128. Definition.—*Disinfection* is treatment with chemicals so as to reduce materially, or eliminate entirely, the bacteria present. When all bacteria are killed, the process is termed *sterilization*. Disinfection is valuable when sewage is to be discharged into a stream, by which the pathogenic bacteria normally present in the sewage may be spread in a dangerous manner. Some preliminary treatment is practically necessary before disinfection to remove as much as possible of the organic matter, the presence of which materially increases the amount of disinfectant needed and reduces the efficiency of the disinfecting action.

129. Chemicals Used.—Many chemicals are available for disinfection, including most of the acids and copper sulphate and the chlorine compounds. In practice, the last are used almost exclusively, because of their effectiveness and relative cheapness. Liquid chlorine, and hypochlorite of lime, commonly called bleach, are the two principal agents used in disinfection, the killing of the bacteria being accomplished by the oxygen released by the action of the disinfectant. As the oxygen is set free, it combines instantly with organic matter present. When bacteria are at hand, the oxygen attacks and kills them. The methods of applying these disinfectants to sewage are the same as those used in purifying water.

Bleach was formerly used a great deal in the disinfection of water and sewage, but it has been superseded to a considerable extent by liquid chlorine because of the difficulty in applying it properly. It is convenient for emergency applications, as it often can be obtained locally, and can be applied without any special equipment.

Acids are generally quite effective as disinfectants, but the cost is high under most circumstances. Copper sulphate has also been used, but it is not nearly so effective as the chlorine

compounds, and is much more costly. Sodium hydroxide and many other chemicals of the same sort, while possessing bactericidal properties, are too costly or are otherwise unsuitable.

SUMMARY OF SEWAGE TREATMENT

TESTS FOR SEWAGE EFFLUENTS

130. Imhoff Glasses.—Odor and appearance are valuable indications to a trained observer as to the condition of sewage or sewage effluents, but in addition certain more specific tests are necessary. It is possible to determine roughly the amount of settleable solids in sewage by means of *Imhoff glasses*. These are conical vessels of glass, about 17 inches high and holding 1 liter. The lower part of the cone is graduated to cubic centimeters, and the percentage of solids settling in any given time may be determined by noting the graduation mark to which the sediment reaches and dividing this value by 10. The results, however, are not very accurate on account of uneven sedimentation and compaction of the solids in the cone.

131. Methylene Blue Test.—The methylene blue test is a very simple test to make, and, in connection with other tests, it furnishes valuable information in regard to the stability of the sewage. A sample of the sewage effluent is placed in a glass-stoppered bottle, and a small quantity of methylene blue added. The treated sample is then kept either at 20° centigrade or at 37° centigrade to determine its stability. If the blue color is retained for 4 days at 20°, or for 2 days at 37°, the sample is deemed to be stable. If the sample is putrescible, the blue color will disappear.

USES OF VARIOUS METHODS OF TREATMENT

132. Problem to Be Solved.—It has been pointed out that the problem involved in sewage disposal is to get rid of a large quantity of water containing, both in suspension and solution, a small amount of organic matter, and also a small amount of

inorganic matter. The water must sooner or later be turned into a watercourse, as drainage always finds its way into some stream. The organic matter, unless given an opportunity to oxidize, becomes putrid and odorous and may contain, even when apparently inoffensive, disease germs. A method must be devised whereby this small amount of putrescible matter in sewage can be completely, cheaply, and thoroughly treated, so that the water can be discharged safely into a stream.

The particular method of treatment to be adopted will depend almost entirely on local conditions. As previously stated, if a large stream or body of water is available, dilution may meet all the requirements of sanitary disposal of the sewage. Where flotage from the sewage may be deposited along the shore and cause nuisance, or where the dilution is not quite ample, removal of the suspended grosser solids, as by screening or simple tank treatment, may be sufficient. In other places, more expensive treatment may be needed.

133. Disposal by Dilution.—Direct discharge into streams, lakes or other bodies of water may be practiced most cheaply, and without offense, if the dilution is in a ratio of 1 part of sewage to at least 40 parts of water; if the currents or tides carry away the organic matter, so that no deposits are formed; and if there is no danger of spread of disease.

134. Tank Treatment of Sewage.—For most places it is necessary and economical to give the sewage some primary or preparatory treatment whereby a part of the matter in solution and suspension is removed. By sedimentation and bacterial action in sedimentation, septic, or Imhoff tanks, the grosser solids are removed, and a partial clarification of the sewage accomplished. The treatment is not complete, however, and further treatment is necessary to oxidize finally the unstable elements in the sewage.

Chemical treatment is not very satisfactory, and is applicable to but few places. The cost is great, and only a slight degree of purification is obtained. In addition, there is a large volume of sludge, the treatment of which is difficult and expensive. Although activated sludge treatment is used to some

extent in large cities, it is unlikely that this method will soon be applicable to small cities, or even to many cities of medium size.

135. Secondary Treatment.—Preliminary treatment effects only a partial purification of sewage; in fact, it does little more than break down the compounds making up the organic matter in the sewage. Oxidation must take place in order to bring the sewage to such a point that it will not cause nuisance. The methods of secondary treatment most generally used to accomplish oxidation are sand beds, contact beds, and trickling filters.

Partly clarified sewage may be applied to sand beds at the rate of about 75,000 to 100,000 gallons per acre per day. The resulting effluent is practically pure, clear, and sparkling. Sand filtration is very effective and satisfactory, especially in those sections where the soil is naturally so open and porous that extensive areas are available at a low cost. Underdrainage is necessary; no vegetation is allowed; and the soil receives all the sewage it can take. Regular periods of dosing and rest must be given. Since 1,000 persons per acre is about the maximum, the use of this method is limited to small cities.

Contact beds will treat settled sewage at a much higher rate than will sand beds, but the resulting effluent is not so pure. However, such treatment may produce results sufficiently good to allow the sewage to be discharged into a stream.

Where trickling filters are used, the sewage is sprinkled over a bed of broken stone, instead of being held in contact with it. Still better results are obtained, and a much higher rate of treatment is possible than with contact beds. Purification is accomplished by the bacteria on the surface of the stone particles, as the sewage trickles down through the interstices of the bed.

136. Standard Practice.—What might be called standard practice in present day sewage treatment consists of preliminary or primary tank treatment, with secondary treatment by means of trickling filters.

IRRIGATION

INTRODUCTION

1. In spite of the large areas that, in certain sections, are already irrigated for the purpose of cultivation, it may be affirmed that irrigation as a science is still in its infancy in the United States. There are some countries, notably India, where it has not only been practiced from a remote period, but where also its principles are well understood, and its operations are carried on in a systematic manner and frequently on a gigantic scale. In those countries, irrigation has passed beyond the experimental stage, and has taken its place as an established branch of agricultural science. In still other countries, such as Mexico, although irrigation is not practiced on the large scale nor in the systematic manner prevailing in India, yet its advantages are thoroughly understood and appreciated; and it is very interesting, in traveling through the semiarid agricultural districts of that country, to note the skilful manner in which the small cultivators have taken advantage of every little streamlet to store and distribute the few scanty drops of water necessary to secure their otherwise precarious crops.

2. Necessity of Water in Raising Crops.—While it is pretty generally understood that water is necessary to the growth of plants, and that vegetation thrives with moisture and languishes in drought, it is not, perhaps, so generally understood why this is the case. Briefly, the reasons are the following: Plants require a certain, and often a very considerable, proportion of water as one of their constituent parts. This water they absorb from the soil by means of

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their roots. The soil must contain a certain percentage of moisture—perhaps from 5 to 10 per cent.—before it can yield up any to the plant. On the other hand, the plant is constantly giving off moisture by evaporation from the leaves, and when the amount thus lost exceeds the amount taken up by the roots, the plant droops and dies. Besides, plants cannot assimilate their food in the solid state, but only in a fluid condition. It is necessary, therefore, that the substances they take from the soil should be in solution, and that the soil should contain enough water to dissolve those substances.

3. Natural Irrigation.—In those regions of the United States that enjoy a normal rainfall, comprising, as a rough approximation, all the territory lying east of the ninety-seventh meridian, the natural precipitation is depended on exclusively to supply the necessary solvent for plant food. That this precipitation may satisfactorily accomplish the result, two conditions are evidently necessary: first, the annual amount must be sufficient, and second, it must be distributed at proper seasons, so as to be timely as regards the needs of vegetation.

In the regions enjoying an average yearly rainfall of 30 inches or more—which would be much more than sufficient for plant growth if falling opportunely—there may still be serious damage done by drought, even to the extent of losing certain crops, from the fact that rain is lacking just at the right time. Even in the most favored localities, the element of uncertainty introduces a most dangerous factor in all agricultural pursuits. On the other hand, serious loss is often sustained by excess of rain when not wanted.

4. Artificial Irrigation.—The amount of water actually needed for the growth of crops is relatively very small. As a rough general average, it may be stated that between 12 and 20 inches per year, spread at proper seasons over the duly prepared surface of the area under cultivation, is sufficient. In order that these conditions may obtain, it is necessary to have under control and in store a volume of

water sufficient to cover the entire cultivated surface to the depth of from 1 foot to 2 feet, allowing for loss by evaporation and other causes, and to have also facilities for spreading this water over the given area as wanted. It is clear that this makes it necessary to provide for a considerable excess over the amount absolutely needed for irrigation, in order to provide for losses. But given a sufficient quantity of water, and adequate means of controlling its use, artificial irrigation is superior to natural, in that it gives the proper quantity of water to the plant, at the proper season. Indeed, the claim seems justified that, instead of artificial irrigation being a substitute for rain, rain is an imperfect substitute for artificial irrigation.

5. Commercial Value of Irrigating System.—A study of the cost and possible returns of irrigation work is instructive and essential, in order that the financial possibilities of any projected irrigation system may be determined. In the last censuses of the United States, statistical tables have been compiled that furnish an excellent basis for estimating the value of irrigation systems. The results vary greatly according to the region of the country considered, the nature of the soil, and the kind of crop that may be grown. Returns from irrigation in Southern California and in Arizona, where valuable crops of grapes, oranges, and other citrous fruits mature luxuriantly, and where four or five crops of hay may be cut in a season, are far greater than in Montana and in Wyoming, where only hardy grains and hay can be grown, and only one crop of the latter is obtained in one season.

The average cost of construction of irrigation works in 1902 was $9.84 per acre irrigated. The value of crops grown and marketed averaged, for each acre-foot of water used, in Montana, $18.42; in Utah, $6.34; in Arizona, from $10 to $30; in Northern California, from $10 to $20; and in Southern California, from $50 to $240. The average size of an irrigated farm in 1902 was 71 acres. As to the extent of irrigation, there were 9,487,000 acres irrigated in the West

in 1902, the total cost of construction being $93,320,000. The water was distributed through 33,415 systems, which included about 59,300 miles of main canals.

The average first cost of water per acre, as shown by the census, was $7.80. The average value of the products from this land per acre was $14.87. The average annual rental paid for the water per acre was $.38. The average value of irrigated land per acre was $42.53. Before irrigation was introduced, the average value of the same land was from $2.50 to $5 per acre. This shows the value added to land by the mere addition of water.

6. Irrigation Only One Factor in Cultivation. Mere irrigation must not be exclusively depended on to render arid soil productive, although in any case it may cause a temporary fertility at the start. Applied alone, and injudiciously, it may even decrease the fertility. It is only one of several factors in the reclaiming of otherwise uncultivable soil. It must be combined with a proper selection of crops suited to the particular soil; with proper underdrainage, natural or artificial; with cultivation and mellowing of the soil; with mulching; and in many cases with fertilizing. In a word, every other resource of the agriculturist should be brought into action, just as would be done in ordinary farming when no artificial irrigation is practiced. A neglect of those precautions has, no doubt, often led to disappointment and loss of faith in irrigation.

WATER FOR IRRIGATION

QUANTITY AND QUALITY

7. Units of Volume Used in Measuring Water. For irrigation purposes, the volume of water may be measured in cubic feet. There is, however, another unit that is very commonly employed, and is very convenient; it is called the **acre-foot,** and is the amount of water that will cover to a depth of 1 foot a flat surface having an area of 1 acre. As an acre contains 43,560 square feet, an acre-foot is equivalent to 43,560 cubic feet. A unit often used in the Western American states is the miner's inch (see *Hydraulics*).

8. Duty of Water.—The **duty** of a water used for irrigating a given area is the relation between the quantity of water used and the area irrigated. If water is measured in acre-feet, and area is measured in acres, the duty is expressed in acres per acre-foot, and is obtained by dividing the area irrigated, in acres, by the number of acre-feet used. Thus, if 4 acre-feet can irrigate 1.75 acres, the duty is $1.75 \div 4 = .4375$ acre per acre-foot. Occasionally, the duty is expressed as so many acre-feet of water required per acre of irrigated area, but this is really the reciprocal of the duty, not the duty itself.

9. Sometimes, the duty of water is stated as so many acres to the cubic foot per second. In this method of measurement, the water used during the irrigation period is assumed to be flowing at a uniform rate; the number of cubic feet per second is therefore obtained by dividing the total number of cubic feet used during the irrigation period by the number of seconds in that period. If the number of acres irrigated is divided by the number of cubic feet of water used per second, the result will give the duty expressed

in acres per foot per second. This method of expressing duty, although common, is in many respects unsatisfactory, and is disapproved by the best authorities on irrigation.

10. Duty per Acre-Foot.—"Assuming an average depth of 4 inches of water as sufficient to thoroughly soak the soil, this is equivalent to $\frac{1}{3}$ acre-foot per acre. An average crop requires from two to four waterings per season. Assuming three as the mean, then, at the above rate, 1 acre-foot will be required per season to irrigate 1 acre. Practice, however, clearly indicates that this theoretic amount is too low. Experiments conducted in Wyoming indicate that 12 inches in depth for potatoes to 24 inches in depth for oats is sufficient to mature crops. In Idaho, the depth of water generally used is about 24 inches; while in Montana, from 15 to 18 inches is believed to be sufficient. These indicate volumes ranging from $1\frac{1}{4}$ acre-feet in Montana to 2 acre-feet in Wyoming and Idaho. Measurements on several canals in Colorado show that from 18 to 24 inches in depth of water is required, or from $1\frac{1}{2}$ to 2 acre-feet per acre. Experiments conducted by Mr. Samuel Fortier in Utah indicate that a depth of 24 inches is required for tomatoes, while potatoes yield abundantly with a depth of 17 inches, onions with a depth of 36 inches, strawberries with 27 inches, and orchards with 12 inches. In India, the average duty of water, figured from the volume entering the distributary head, has been found to vary from $\frac{1}{3}$ to $\frac{1}{2}$ acre per acre-foot.

"In estimating the duty of water stored in a reservoir, allowance must be made for the losses due to evaporation and absorption in conducting the water to the fields. As this averages 25 to 50 per cent., it follows that, where a duty of 1 acre per acre-foot is possible, $1\frac{1}{4}$ to $1\frac{1}{2}$ acre-feet per acre must be stored in the reservoir; and where $\frac{1}{2}$ acre per acre-foot is the duty, $2\frac{1}{2}$ to 3 acre-feet per acre must be stored."—*Wilson.*

11. In order that all consumers may not demand at the same time the use of the water flowing in the canals, it is found necessary to apportion the service periods for each

irrigator through the 24 hours of each day and through the season in such a manner that there will be a system of rotation employed in opening the heads of the ditches of the various users. The best method of arranging the rotation is to divide the main canals into a number of sections and the various distributaries (distributing canals) into sections, and to allow the water to only some of the sections at a time.

12. Irrigating Periods and Quantity of Water Required.—The length of time during which water is applied to land in the course of a season is called the **irrigating period.** Each such period is subdivided into several **service** periods.

"In Colorado, alfalfa and clover are irrigated twice in a season, once in May and once in June, to a depth of 6 inches for each period; wheat and oats are irrigated twice, once in June and once in July, to a depth of 9 and 6 inches, respectively. Meadow or native hay requires considerably more water; there are usually two service periods, each of which lasts several days, the water being allowed to run in a small quantity during that time. The first is usually in May, and is about 2 inches in depth for a week; the second in July or August, of about the same amount. Since the application of water is generally followed by a temporary checking of the growth of the plant, the method preferred in the arid region seems to be to give thorough rather than many irrigations; in other words, to have two ample rather than four to six small services. In general, it may be stated that two or three service periods, varying in depth from 3 to 6 inches, are employed in Colorado, and that the irrigation period extends from May to September—123 days. In Utah, the practice seems to be to employ a much larger number of service periods—from three to five on grain crops, of 2 to 3 inches in depth each—the water running 12 to 15 hours per service period, and the irrigation period extending from June to August, inclusive. On vegetables, as many as six or ten service periods are employed, each lasting from 3 to 6 hours, during June to August, inclusive. The irrigating

period in the majority of Western states averages from April 15 to August 15, or about 120 days; while the service period varies from 3 to 15 hours in length, according to soil and crop, and there are from two to eight such service periods in an irrigating period. In India, there are from three to five service periods, making up an irrigating period of from 100 to 130 days' duration."—*Wilson.*

13. Quality of Water.—Evidently it is not necessary to make so close a scrutiny of the quality of the water used for irrigation as for a domestic supply. Indeed, some waters totally unfit for domestic use from the presence of a large amount of organic matter are thereby rendered peculiarly favorable for irrigation, owing to the fertilizing properties of the substances held in suspension. But, although this suspended material is frequently beneficial to the land, it becomes sometimes very troublesome, by obstructing channels and waterways, and filling up reservoirs, particularly when the entrained silt is composed of mineral substances.

A very important factor in the value of water for irrigation is its temperature. The warmth imparted by water of a relatively high temperature is of itself frequently sufficient to greatly stimulate plant growth.

14. Drainage Connected With Irrigation.—In order that the territory operated on may derive the full benefit of irrigation, it is necessary that facilities be afforded for the removal of the surplus water after the soil has been thoroughly saturated. No benefit is derived if the soil is allowed to become water-logged. It is necessary that the water applied should slowly pass through the ground, and not remain on it until removed by evaporation.

Drainage, like irrigation, may be either natural or artificial. Frequently, the character of the soil is such that the drainage takes care of itself; this occurs when the ground is underlaid by a porous substratum; but at other times artificial drainage should be resorted to.

15. Drainage Necessary to Prevent Alkalinity. In many parts of the Western American states the presence

of "alkali" is a serious impediment to the growth of crops. The presence of alkali is manifested by a white efflorescence on the surface of the ground, consisting chiefly of chloride of sodium (common salt), sodium carbonate (sal soda), and sulphate of sodium (Glauber's salt). The effect of these salts on vegetation is most pernicious, particularly the sodium carbonate, known as "black alkali." The deposit of alkali on the surface of the ground is due to the evaporation of considerable quantities of water containing these salts in solution.

The best preventive of the formation of alkali is found in underdraining the soil. In regions where alkali prevails, soils not naturally underdrained should, if possible, be avoided, and only those that have natural advantages in this respect should be selected for irrigation. If the difficulty cannot be avoided in this way, it must be combated by artificial drainage.

16. Other Remedies for Alkali.—Although underdraining is the most radical and effective means of combating alkali, there are other remedies that may be employed, either alone or in connection with drainage. Mulching the soil, or giving it a top dressing of any kind suitable to shelter it and impede evaporation, is sometimes a valuable aid. The evil effects of black alkali are greatly diminished by the use of gypsum as a top dressing, but it appears to be thoroughly effective only when the soil is also underdrained. Sometimes, when there is an abundance of irrigating water, the deposit may be washed off the surface by flooding it, and rapidly drawing off the water before it can soak into the ground.

Some crops are less injured than others by alkali. Alfalfa, or lucerne, seems to be the least affected by it, and can be grown to advantage when other crops would fail.

It general it may be said that, in order to cultivate successfully ground afflicted with alkali, recourse should be had to underdraining, the use of a minimum amount of water in irrigating, cultivation, and mulching, and the application of plaster of Paris, or gypsum.

17. Silt and Sedimentation.—The rivers of arid regions carry in suspension, especially during flood times, great volumes of silt. This is derived chiefly from the erosion of the alluvial banks of the stream. As this enters the canals, the velocities of which are slower than those of the rivers, much of the suspended matter is deposited near the heads. The same is true when the matter enters the upper ends of storage reservoirs. The result is to diminish the volume of the reservoir, or the discharging capacity of the canal, as the case may be. These facts must be taken into consideration, therefore, in designing canals, in order that they may be given such grades that they will retain the finer silt in suspension until it reaches the land, where it is valuable because of its fertilizing properties. Also, canals should be so designed that as large a portion of the silt as possible shall be deposited within a short distance of the head, where provision may be made for removing it by dredging or by scouring.

Some of the Western streams carry vast amounts of sediment in suspension. On the Rio Grande it has been found to be as high as from one-fourth to one-half of 1 per cent. of the volume of the flow. On the American River in California, a depth of nearly 10 feet of wet silt was deposited in the reservoir in 1 year. In 12 years, 900 acre-feet of sediment was deposited in the Sweetwater Reservoir, California. On the Gila River, Arizona, sediment carried in suspension averaged about 10 per cent., and the amount of solids 2 per cent.

Silt-laden water has a very high fertilizing value. In the Moselle valley of France, barren land that was absolutely worthless without fertilization produced two excellent crops after irrigation with water heavily laden with alluvial matter. The turbid waters of the River Durance in France bring prices many times greater than that paid for the clear, cold water of some neighboring rivers.

SOURCES OF SUPPLY

18. There are two sources of supply that are commonly looked for in studying an irrigation project; namely, surface and ground waters. Generally speaking, all that has been said on this subject in *Water Supply* applies to irrigation. There are some points of difference, however, that must be noted. In the first place, as has already been mentioned, the question of hygienic quality is virtually eliminated from the problem. The chemical character of the water has, it is true, some bearing on its fitness for irrigating purposes, as being favorable or the reverse to the formation of alkali; but, broadly speaking, neither biological nor chemical examination plays any prominent part in this branch of hydraulic engineering. In the second place, since irrigation is mostly practiced in districts where the rainfall is abnormally small, general rules are less applicable, as regards the supply derivable per square mile of drainage area, than for districts of average rainfall and evaporation. More attention must be paid and more weight given to gauging, measuring, and observing, at least until a good general knowledge has been obtained of average conditions in the arid and semi-arid districts that form the principal field of irrigating operations; and more pains must be taken to secure accurate results, on account of the small quantities dealt with. Each case will be more or less a special one, requiring special study.

SURFACE WATER

19. Preliminary Observations.—The first question to be decided will be the amount of water required. In this estimate, it will be well to make very liberal allowances for losses by waste, by evaporation, and in transmission. Suppose that in a given project it is thought that a yearly quantity of water, sufficient to cover the whole area to be irrigated to a depth of 24 inches, is necessary to include all items of use and loss. Then, if the given area contains 10 square miles, or 278,784,000 square feet, the yearly

amount of water required, in cubic feet, is this area multiplied by 2, or 557,568,000 cubic feet. If, therefore, it is desired to secure this amount of water by the aid of a storage reservoir, formed by building a dam across a certain stream, it is necessary to ascertain if the drainage area situated above the proposed dam, combined with the minimum available rainfall, is sufficient to afford the required quantity. It may be here remarked that, in the study of the quantities necessary for irrigation, the engineer is not bound so rigidly as in cases of water supply for communities. In the latter case, any failure in the daily quantity of water furnished leads to dangerous, or, at least, very inconvenient, results. Obviously, the failure to supply the full quantity that would be required for irrigation cannot be followed by so serious consequences as will a water famine in a populous city.

It has been explained in *Water Supply* that the available yield of any particular watershed or drainage area is not given by its area multiplied by the depth of yearly precipitation. A large percentage of this amount is lost by evaporation, by absorption, and from other causes. The remainder, which finds its way to the stream to which the drainage area is tributary, and which is known as the **run-off,** is all that is available for storage.

20. Study of Watershed.—The first thing that must be done in the study of an irrigation project derived from surface water is to collect data. These will consist of a survey of the watershed, gauging the flow of the stream, and measuring the precipitation or rainfall. The survey will be conducted on the same principles as the survey for any water supply. As from its nature it must be an approximate one, no time should be wasted in unnecessary refinement of instrumental work. A plain chain-and-compass survey is all that is needed.

The traverse survey should follow the crest of the drainage basin tributary to the reservoir site, in order that it may develop as nearly as possible the area of the catchment basin. Such a survey is unnecessary if the topographic maps of the

United States Geological Survey cover the area under consideration, in which case the area can be measured from those maps with all necessary accuracy. In like manner, the records of the United States Weather Bureau should be consulted for rainfall data, thus avoiding the necessity of making measurements. Finally, valuable data for determining the discharge of the water can be obtained from the records of the hydrographers of the United States Geological Survey, which may show that the stream itself or some of its neighboring tributaries draining like areas have been gauged.

21. Rainfall.—The subject of rainfall is fully treated in *Water Supply*, Part 1, where methods are given for gauging rain precipitation. Here, only that part of the subject that bears directly on irrigation will be considered.

The amount of rainfall is an extremely variable quantity: it varies not only with the locality, but also with the seasons, and often it is greatly different in two regions that are not far from each other. In the Western American states, it is much less in the flat and arid plains than on the high and well-wooded mountains that may separate them. In and near Sacramento, California, it is 15 inches per year, while at a short distance east, on the crest of the Sierras, it averages 50 to 60 inches. Still a little farther east, but at considerable altitude, in Nevada, the precipitation is as low as 5 to 10 inches. Near Yuma, Arizona, the precipitation during the growing season is only about 1 inch per year, yet not far off, near Prescott, it is 8 inches. The precipitation has been found to increase with altitude in the Sierras of California at the rate of about 6 inches per thousand feet increase in altitude. In the upper Missouri and Yellowstone valleys in Montana, the average annual precipitation is from 12 to 20 inches. Yet, of this amount, only about 5 inches falls during the growing season. In the Platte and Arkansas valleys of Colorado, as much as 8 inches falls during the irrigation season out of an annual precipitation of 15 inches.

22. The average annual precipitation is but an imperfect guide in determining the amount of water available for irrigation, if the water is to be used as fast, or nearly as fast, as obtained. Under such circumstances, the important factor is not the average precipitation, but the precipitation during the season of crop growth. The average annual amount over the northern Pacific coast would produce good crops, if it fell during the irrigating season, but the distribution of the precipitation is such as to materially affect its value for this purpose.

In connection with the water available for storage, however, the average yearly precipitation is of importance. Knowing the average amount of precipitation over the area of a basin tributary to a storage reservoir, it is possible to estimate with some degree of approximation the total volume of water available.

23. Exceptional conditions, such as heavy showers and snow melting, must be very carefully considered. Sudden melting of snow in the mountains after warm rains may produce disastrous floods. For example, the average rainfall near Yuma, Arizona, is but 3 inches; yet, single rainfalls of such violence as to amount to $2\frac{1}{2}$ inches in 24 hours have been recorded. In San Diego, California, where the average rainfall is 12 inches, a storm amounting to 13 inches in 23 hours and aggregating $23\frac{1}{2}$ inches in 54 hours has been recorded. Such storms may fill reservoirs to overflowing, cause the destruction of the dams, and ruin the irrigation system. Salt River, Arizona, has an average annual discharge of about 1,000 cubic feet per second. Its average flood discharge is about 10,000 cubic feet per second; yet, one storm is on record that caused a discharge of 300,000 cubic feet per second, and of sufficient violence to wash away bridges and heads of canals.

Another important condition is the suddenness of great rainfalls. Storms have been recorded with precipitation as high as 5 inches in 1 hour. In Baltimore, Maryland, a storm is on record lasting for 2 hours with a rate exceeding 6 inches per hour. The destructive effects of such storms

cannot be overestimated, and must be considered in the construction of water-supply systems.

24. Run-Off.—While the average and the periodical amount of precipitation are important factors in the designing of an irrigation system, the most important factor in determining the water supply available from any watershed is the *run-off*. As stated in *Water Supply*, the **run-off** is the quantity of water that flows off in a given time from the surface of the land. It is determined by gauging the flow in streams, and includes not only the water that flows from the surface after rainfall, but also that derived from springs, etc.

The amounts of run-off on various catchment basins (watersheds) in the arid regions of the United States have been determined and published by the hydrographers of the United States Geological Survey, whose publications on the subject contain a great deal of valuable information, and should, whenever possible, be consulted for the determination of the run-off of any watershed that it is proposed to use for irrigation purposes.

The run-off has been found to vary between .26 cubic foot per second per square mile per year on the headwaters of the Gila River, Arizona, to .06 cubic foot per second per square mile per year in the flat lands near its mouth. It ranges all the way up to 2.18 cubic feet per second on the steeper slopes of the streams of the Sierras of California.

25. Gauging the Flow of Streams.—Streams are generally gauged by the erection of a dam, in which an overflow weir is placed, as described in *Hydraulics*. In some cases, the construction of a dam and weir is somewhat difficult, because the dam must be made water-tight, so that the entire flow of the stream will pass over the weir.

When great accuracy in the quantity of discharge is not required, or where the conditions under which the observations are made—such as possible leakage around the dam, inaccuracies of measurement, or uncertainty in regard to the velocity of approach—make the result doubtful, the following formula, in which a mean value of the coefficient of discharge

has been used, is simpler than the more accurate formulas given in *Hydraulics:*

$$Q = \tfrac{10}{3} l H^{\frac{3}{2}}$$

Daily observations should be recorded for at least a year, in order to obtain even an approximate knowledge of the normal flow of the stream. The time required to get an intelligent idea of the run-off and general character of a given watershed, by the above observations and those of the rainfall, constitute an embarrassing delay in districts where such observations have not already been carried on for a considerable period before it is desired to begin work. Unfortunately, it is very seldom that systematic records are kept in advance of the requirements.

26. Gauging the Flow With Current Meter.—Although the method of weir measurements is the best adapted to continuous observations, there are many cases in which its use is impracticable, or at least too difficult and expensive, and in such cases one of the other methods described in *Hydraulics* must be used. Of these methods, the one by the current meter is one of the best. With the improvements recently introduced by the engineers of the United States Army and the United States Geological Survey, this instrument gives results almost as accurate as those obtained by weir measurements.

In order that current-meter measurements may be made with accuracy, a straight and smooth bedded stretch of the stream must be selected, one in which there is little change in the slope or form of the bed for a distance of at least 100 feet above and below the gauging station. The cross-section of the stream must be carefully measured, the depth being taken at least every 10 feet. Daily observations should be made for a long period—a year or longer, if possible—so as to obtain good average values. The meter should be read at several depths for each of the different stages of the stream.

27. Measurement of Evaporation.—In the dry regions, where irrigation is mostly practiced, loss by evaporation is often a serious matter, especially for calculating

the proper capacity of storage reservoirs. In sections enjoying an abundant rainfall, this item of loss is rarely given much attention, since it is admitted that the rainfall on the surface of a reservoir will compensate for the loss by evaporation. Against this view, however, must be set off the fact that during the rainy season the reservoir is likely to be overflowing, when the benefit of any compensating precipitation is consequently lost.

It is very difficult to estimate in advance the probable loss by evaporation in a storage reservoir, because any experiments carried on in the stream that it is proposed to dam will be conducted under conditions different from those that will obtain in the reservoir itself. Evaporation is greatest in warm weather and during the prevalence of high winds. It varies with the character of the body of water from which it proceeds, as it is less in a deep reservoir than in a shallow one, and less in still than in running water. In the semiarid regions, evaporation will probably average from 3 to 5 feet in depth from the surface of a reservoir during the year.

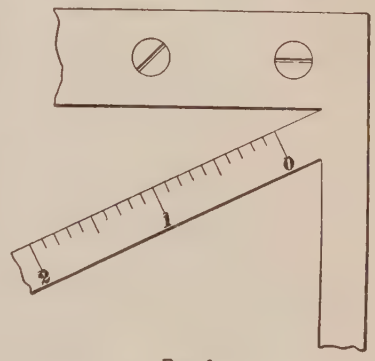


FIG. 1

To measure the evaporation from a given body of water, a simple apparatus is used by the United States Geological Survey. It consists of a pan of galvanized iron, 36 inches square and 10 inches deep, floated in the body of water of which the evaporation is to be measured, and filled with water to within a few inches of the top, care being taken to prevent water from washing in or out of it. A scale, Fig. 1, is placed obliquely in the pan, so that small vertical distances may be rendered appreciable by exaggeration.

28. Evaporation in any locality may be determined as above described. The records of measurement of evaporation

by the hydrographers of the United States Geological Survey and others should be consulted when possible, as they may furnish more valuable data than could be obtained by measurements extending over a limited period of time. In the arid regions of the United States, the monthly evaporation has been measured in many places, and is found to have a wide range. At Salt Lake City, Utah, it amounts to 40 inches per year and ranges from 1 inch in January to 7 inches in August. Near El Paso, Texas, the annual evaporation is 92 inches, ranging from $2\frac{1}{2}$ inches in January to 1 foot in July. There is a perceptible though small amount of evaporation, about 6 inches, from snow and ice. In the arid regions, water is used for irrigation between the months of May and August, during which time evaporation is greatest, and as the precipitation is slight the losses sustained in this way are not compensated for by any corresponding rainfall.

29. Absorption.—The losses due to **absorption**—that is, to seepage and percolation from canals or reservoirs—are very considerable in amount, and must be added to those resulting from evaporation. In some localities, the losses by percolation have been found to be 25 per cent. of the rainfall, while the losses from evaporation were 75 per cent., the entire rainfall being thus lost. In sand, the losses by percolation are necessarily much greater than in earth or clay. By measurement, the losses due to absorption in a canal have been found to average about 1 cubic foot per second per linear mile. They are greater in new canals or in sandy soil and less in old canals, the beds of which have been made less pervious by deposition of sediment. In long canals, the losses by absorption in some years have been found to be as high as 60 or 70 per cent. Seepage into canals and reservoirs replaces to some extent the loss from evaporation and percolation.

GROUND WATER

30. Sources of Ground Water.—It has already been shown that a portion of the water that falls on the surface of the earth in the form of rain runs off rapidly into the rivers and streams, and is conveyed by them ultimately to the sea; while another portion sinks into the ground, and although this also is constantly seeking outlets and lower levels, by virtue of its weight, yet it moves so slowly, owing to the resistance to percolation offered by the media through which it passes, that at any given moment it may be considered as stationary.

During past ages, the water, constantly falling on the earth's surface and slowly sinking through deeper and deeper strata, has finally accomplished such a degree of saturation of the earth's crust that in almost all districts a permanent water-table has been established at a greater or less depth below the surface, and not varying very materially with the seasons. Even very arid districts, almost or wholly deprived of rainfall, may yet possess a store of underground water, which has slowly reached them from distant and more favored regions. We may thus consider the earth as forming a vast storage reservoir, in which the rainfall of many ages has been impounded. Should this supply be suddenly exhausted —were such a thing possible—it would doubtless require many centuries of subsequent rainfall to resaturate the earth's crust to the same degree that obtains at present.

The way in which this water is utilized by the construction of wells was explained in *Water Supply*. In some cases, ground water may be obtained by cutting tunnels intercepting subterranean streams, or by building diaphragms or dams across such streams, so as to force the water to the surface.

31. Pumping From Wells—Windmills.—Water is raised from wells by pumping, as explained in *Water Supply*. If the water is to be delivered above the level of the well, a force main is necessary. When the water from only a

single well is to be pumped and forced through a main, the problem offers no difficulty; when, however, a "gang" of wells is to be pumped, the problem is rendered more complicated. If the water rises nearly to the surface, say within 20 feet, and is not subject to fluctuations materially increasing this depth, all the wells may be connected with one general suction pipe operated by a single pump. Otherwise, a central station will be required, furnishing power for as many lifting pumps as there are wells, which will deliver their water to a common suction main.

32. The best power to work the pumps depends on circumstances. Whenever a constant water-power can be obtained, it will always be preferred, on account of its great economy. The principal objection to water-power is its liability to fluctuations.

Steam will be found, generally speaking, the most trustworthy source of power, and in some cases the engines and boilers may be advantageously located near the point where fuel is most cheaply procurable and where the power may be transmitted by electricity to the pumping station.

In the Far West, where transportation rates are high and all kinds of fuel must be brought long distances, gasoline is found to be one of the most economic fuels, and gasoline engines are in great favor for pumping.

More recently, electric power has been found most economical as a pumping agency, especially where it may be generated from water-powers located at reservoir sites or on running streams.

33. Windmills are employed for works of small magnitude; they are cheap, and give good results when properly used. There are many varieties of windmills on the market. The work that they will perform depends on the force and steadiness of the wind and the size of the wind wheel. In order to give security to the water supply provided by pumping with so erratic a motive power, sufficient storage capacity must be provided to hold the water pumped during the night time, and at such other times as it may not be used in

irrigation. The surplus water may be stored in one of the wooden or metal water tanks supplied by windmill makers, or in a reservoir excavated in high land and suitably lined with wood or cement, or clay and gravel.

The cost of operating windmills for a 25-foot lift, including interest charges, etc., ranges from 7 cents per hour for a 10-foot wheel to 24 cents for a 16-foot wheel, and 43 cents for a 25-foot wheel. A windmill is about 50 per cent. cheaper than a steam pump, but is relatively more expensive to install because of the necessity of providing storage. Windmills cost from $50 to $400, according to the size of mill and type of pump. The larger pumps will discharge from 250 cubic feet per hour for a 5-inch pump to 650 cubic feet for an 8-inch pump. A 12-foot steel mill will furnish about 1 horsepower in a 20-mile wind, and a 16-foot mill will furnish 1½ horsepower in a 20-mile wind.

WATER STORAGE

RESERVOIRS

34. Storage Reservoirs.—Having decided on the quantity of water required and the stream to be used, it is next in order to consider how much of the yield of the stream must be impounded in the storage reservoir.

The general subject of storage reservoirs has been treated in *Water Supply*. Owing, however, to the somewhat different services for which they are intended, there will doubtless be some difference between the functions of a storage reservoir for irrigation purposes and one intended for a public water supply. The draft made on the former will vary from year to year according to the wetness or dryness of the seasons, while the annual supply that must be furnished by the latter will be nearly constant. Irrigation reservoirs will also be depended on to furnish large quantities of water during short periods of time, thus making it necessary that they should be equipped with appliances for

meeting this heavy demand. With public water-supply reservoirs, on the contrary, the demand varies but little from day to day; hence, it is not necessary to provide them with capacious outlets, except such as may be required, in case of an emergency, to empty them quickly.

35. Location of Storage Reservoirs.—If, for distributing purposes, the storage reservoir is to be connected directly with the canal, flume, or pipe line by which the water is distributed to the territory to be irrigated, it should be placed as near as possible to the district to be supplied, so as to diminish the length of the canal. This plan also reduces considerably the losses due to percolation and evaporation. If possible, the reservoir should be so near the land to be irrigated that the water can be conducted through the entire distance in an artificial channel, in order to reduce percolation losses to a minimum. It may, however; be necessary to discharge the water into a natural drainage channel, and divert it therefrom lower down by a weir and canals. In such an event, the loss by evaporation and absorption will be much greater than when the water flows in an artificial channel having the most favorable cross-section. In the plains of the arid regions, natural lake basins can sometimes be utilized as storage reservoirs, which may then be filled by a canal diverting water from some near-by stream. Sometimes, a trial line of levels or a reconnaissance of the ground must be made to determine whether a proposed reservoir site is far enough above the highest point to be irrigated for the water to reach that point by gravity.

36. After the site has been selected, it should be very carefully surveyed, a map being constructed with contours having an interval of not more than 10 feet, so that the capacity of the reservoir may be determined for any given height. A careful survey must be made of the site of the dam, including borings to determine the nature of the foundation and whether or not the surface is underlaid with porous strata that will lead off the stored water. From such a survey, the exact cross-section of the dam, its cubic contents in masonry

or earth, and consequently its cost, and other engineering data may be determined.

37. Reservoirs for Wells.—It is seldom that the daily yield of a well is sufficient for the requirements of the seasons during which irrigation is practiced. It is generally necessary to provide a reservoir capable of containing a certain reserve that may be drawn on as wanted. If possible, such a reservoir is formed by building a dam across the valley of some stream, by which a combination may be effected, the dam catching whatever flow may come down the stream in the rainy season, as well as the supply pumped up from the well. In the case of springs, the reservoir is sometimes formed around the spring, which feeds it as a fountain does its basin.

DAMS

38. Timber Dams.—In works for irrigation, there will be more frequent occasion to build small, cheap reservoirs than in the case of a public water supply, because such works are often erected by individuals for private use on their own farms. Dams for reservoirs of this kind are usually constructed without the aid of a hydraulic engineer, but there is a right and a wrong way of executing even these small undertakings, and a few words about them will not be out of place.

These dams usually have a limiting height of about 10 feet, and impound not more than 1,000,000 cubic feet of water. Beyond these limits, the danger incurred by imperfect work becomes so great that nothing but the most substantial and scientifically constructed work, such as has been described in *Dams*, should be allowed.

The class of work now under consideration may be built on any one of many designs, which it would be impossible to consider in detail. Briefly, all these designs have for their object the avoiding of stone masonry laid up in cement mortar, which would place the structure in a higher class and consequently require skilled labor for its construction.

The best type for these home-made dams consists of timber cribwork, filled with well-packed stone, and backed on the

Fig. 2

water side by a well-constructed earthen embankment. Loose stones not confined in cribs should be avoided, as they are liable to be carried away by freshets. The cribwork acts as a substitute for mortar, by binding the stones together so that they will act as a whole. Fig. 2 is a section showing the general features and minimum dimensions of a dam of this kind; the dam is 10 feet high to the level of the spillway or overflow. This figure, however, does not show the spillway, which is illustrated in Fig. 3 and explained later. To build such a dam, a good trench is excavated, deep enough to reach a satisfactory foundation, and a pavement of large stones, well-packed in with spalls, is carefully placed on the bottom. On these stones, the cribwork is placed, consisting of

either round or square timber notched and treenailed together. The cribs should be carried well into the bank on each side, and the ends very carefully packed, to prevent water from passing around them.

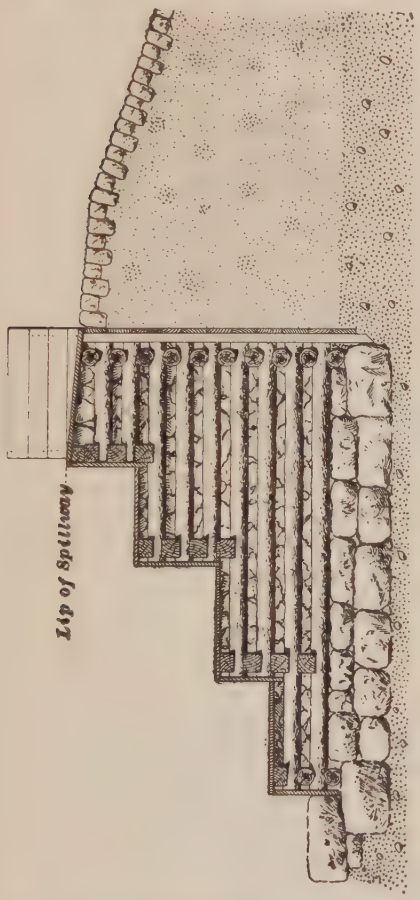


FIG. 3

The cribs are filled with stone of various sizes, tightly packed. The face of the crib on the up-stream side is sheathed with tight planking. Against this sheathing the earthen embankment is placed, the natural surface of the ground beneath it having been roughly excavated, by plowing and scraping, for a foot or so, according to the nature of the soil. This embankment should be ripraped.

The spillway for these dams should be ample; the length may be about 10 per cent. greater than for the carefully built dams described in *Dams*. At the spillway, the cribs require reinforcing; this is accomplished by extending and stepping them, as shown in Fig. 3. The face timbers should preferably be made square, for the convenience of spiking on the planking with which the face should be completely sheathed.

It must be repeated that Fig. 2 shows minimum dimensions for first-class work of its kind. For ordinary work, it

will be prudent to somewhat exceed them. The thickness of the solid stone and timber work should never be less than the maximum height to which the water may rise behind the dam.

Such dams, when care and judgment are used in their construction, are safe and substantial so long as the timber forming the cribs remain sound. Under ordinary conditions, it will be long before the timber decays. If the work is well done and good material is used, the embankment will in time become so packed and consolidated that, even after the cribs have, through decay, lost much of their original strength, the dam will continue to be a safe structure. Dams, however, of this construction are not to be considered permanent in the same sense as stone dams or even earthen dams with a masonry center wall.

Probably the simplest and best way to draw off the water from a dam of the type just described is by means of a cast-iron pipe running through it and under the embankment. The pipe should be placed, if possible, on the natural surface of the ground, to avoid settlement. If this is impossible, a stone foundation resting on the natural ground should be built under it. Gates or stop-cocks should be placed in the pipe, outside of the dam. Sluice gates, stop-plank, etc. will not be needed in this class of work, if the above pipes and appliances are used.

39. Reinforced-Concrete Dams.—A noteworthy, because comparatively recent, deviation from the usual type of masonry dam is one built of concrete with embedded steel to reinforce it. Temperature changes in high masonry dams cause expansion and contraction, especially in the part above water, thus producing stresses that may rupture the structure. To guard against such rupture, which is usually the result of tensile stress, iron or steel, chiefly railroad rails, is embedded in the upper portion of the dam. Some excellent dams of moderate height have been built of reinforced concrete throughout. By the use of this material, it has been possible to give a slighter cross-section, and in some

cases material is saved by making the dam a hollow shell of concrete heavily reinforced with steel bars.

40. Loose-Rock Dams.—In inaccessible portions of the arid regions, where the only foundation available is rock or hard pan, and where rock is abundant near-by and earth for construction of earth dams is not accessible, some substantial structures have been built of loose rock. The loose-rock type may be especially desirable where the cost of cement is high. Such a structure consists of masses of broken stone, just as it is blasted out of near-by hillsides, the smaller stones being used to fill up the spaces between the larger ones. Side slopes of about 1 to 1 are usual. The top width should be 10 to 20 feet, according to the height, and an ample wasteway should be provided as for earth dams, to avoid the possibility of the crest being topped by overflowing water. One advantage of loose-rock dams is that they may be constructed in flowing water. Another is that a leak is not so serious a menace as in earth or masonry dams.

In building a loose-rock dam, the material should be deposited in layers in such a manner that the extremities of each layer shall be higher than the center. The upper slope should be faced with planks or with iron plates to render it water-tight. To prevent seepage through the dam, a center wall, having the same position as the core of an earthen dam, is usually built; this wall may be made of rubble masonry, of plain concrete, or of boiler plates embedded in concrete.

CONDUITS

41. When a large volume of water is to be conveyed a long distance and distributed in measured quantities over an extensive territory along the route, open canals having a suitable fall are generally employed. Such canals have some objectionable features, one of which is, a considerable loss by evaporation and percolation, the latter being an unknown quantity until the canal is built and put in operation. They must also follow a nearly uniform and easy grade, a condition that makes it necessary either to skirt along the valleys they encounter, thereby greatly increasing their length, or to cross these valleys on aqueducts, which are always more or less expensive structures.

When the volume of water to be conveyed is not very great, canals can be replaced by flumes or by pipe lines. Flumes, being open channels of wood, iron, or concrete, differ from canals chiefly in the material of which they are made. Pipe lines, which may be of either cast-iron, wrought-iron, steel, or wooden-stave pipe, differ from canals and flumes in the fact that they are not confined to a uniform descending grade, but may go up or down hill, and, if necessary, can be laid in the bed of any stream that they must cross.

These classes of conduits will be taken up in order, beginning with open canals.

CANALS

LOCATION AND GENERAL DESIGN

42. Preliminary Surveys.—Surveys of a more or less extensive character are the necessary preliminaries to the design and construction of either a canal or a pipe line; this is especially true of the former, because the course of the canal is confined to narrower limits than that of the pipe

line. Although much of what follows regarding surveys will be common to both, it must be understood throughout that it is a canal line that is especially under consideration.

Such a line naturally begins at some point very near to the stream that furnishes the water, and it will generally follow the same valley for a considerable distance. If time permits, it will be of the highest utility to secure a general survey and profile of the stream itself for the entire distance that the canal follows it, noting all tributaries, falls, and rapids. It will be found that it pays well to make a careful and thorough survey, as more work can frequently be done in a few days with transit, chain, and level than in many weeks with pick and shovel. As far as alinement is concerned, this survey does not call for any great accuracy. The leveling, however, is of much importance; for it must be remembered that in this work very large errors are likely to pass unperceived, and that no line of levels can be trusted that has not been checked. Therefore, when the alinement has been completed and leveled, check-levels should be run back over the entire line; it will not be necessary, however, to verify the entire profile, a check on the benches being sufficient. All important tributaries should also be surveyed, the survey being carried up the valley until an elevation approximately equal to that of the starting point has been reached.

A rough estimate of the length of the canal can be made from this survey, in connection with the topographical notes taken at the same time; then the approximate length, together with the total fall to be overcome, will enable the engineer to make a preliminary design of the section and grade.

43. Trial-Line Location.—A trial line for the canal can now be run. There are several ways in which this can be done. One of the best and most expeditious methods is as follows: Suppose that a grade of 4 feet to the mile is decided on for the slope of the canal. The tangent of the angle corresponding to this slope is $\frac{4}{5280} = .00076$, which corresponds to an angle of nearly 3 minutes. Having

a transit provided with a vertical limb, the telescope is depressed to this angle, clamped, and turned in the approximate direction of the line to be followed. The target of a leveling rod is then set at the height of the telescope of the transit from the ground, which can be sufficiently approximated by holding the rod alongside of the transit and sighting across the wyes. The rod is now taken as far ahead as possible and moved along the ground, up or down hill. At the same time that the rod is moved up and down, the transit may be slightly turned in azimuth to the right and left, until the horizontal cross-hair cuts the target. The foot of the rod is then on ground at the desired elevation; a hub is driven at this point, and the distance from the instrument is measured. The transit is then set up over this hub, and the operation repeated. The relative directions of the lines thus run may be ascertained by the needle, as this will be sufficiently accurate. From time to time, measurements will be taken to convenient stations on the line of the river survey, if such a survey has been made, as a check. It will be well to carry this line along, following all the indentations and tributary valleys. It will be very rare that this line is actually followed by the canal, as it would probably be too devious. Some valleys will be crossed on aqueducts, and some hills will be tunneled; but only by a complete survey can the comparative advantages of alternative lines be compared. When an approximate location of the line has thus been determined, the line should be accurately rerun and leveled over, so as to establish the final location and make a more nearly exact estimate of cost.

44. Grade.—The grade of the canal depends on several conditions. In the first place, the character of the bottom and sides will place certain limits on the velocity of the water, which must be great enough to prevent the deposition of silt, and not so great as to do injury to the canal bed. The grade necessary to maintain the velocity within the desired limits will also depend on the character of the interior surface of the canal, being very much less for a

surface having a smooth lining—of concrete, for instance—than for one merely excavated in the earth. The form and area of cross-section also affect the grade, because they affect the velocity of flow. In general, the grade depends on the velocity, and the latter must be determined first.

Formulas for the flow of water in channels are given in *Hydraulics*. Kutter's formula is applicable in all cases, although, for some special conditions, there are other formulas that are simpler and lead to sufficiently close results.

EARTHEN CANALS

45. Formula for Canals With Earthen Banks.—An approximate formula that may be used for canals with earthen banks in good condition is the following:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{9r + 35}} \qquad (1)$$

in which v = mean velocity, in feet per second;

r = hydraulic radius, in feet, $= \frac{a}{p}$;

s = slope $= \frac{h}{l}$.

Solving formula 1 for s, we get, for the slope,

$$s = \frac{9r + 35}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

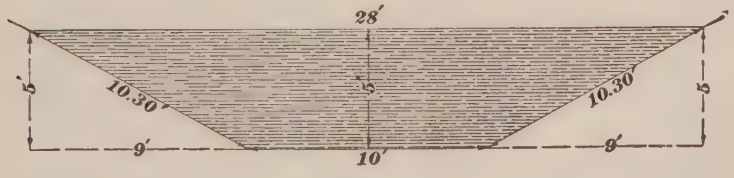


FIG. 4

EXAMPLE 1.—If the fall in an earthen canal having the cross-section shown in Fig. 4 is 5.25 feet per mile, what is the mean velocity of flow?

SOLUTION.—Here $s = \frac{5.25}{5{,}280} = .001$, nearly; also, $r = \frac{95}{30.6} = 3.105$. Then, by formula 1,

$$v = \sqrt{\frac{100{,}000 \times 3.105^2 \times 0.001}{9 \times 3.105 + 35}} = 3.91 \text{ ft. per sec.} \quad \text{Ans.}$$

EXAMPLE 2.—With the same form and dimensions as before, what should the slope be, if the velocity is to be 2.5 feet per second?

SOLUTION.—Substituting known values in formula 2,

$$s = \frac{9 \times 3.105 + 35}{100{,}000}\left(\frac{2.5}{3.105}\right)^2 = .0004. \quad \text{Ans.}$$

46. Limiting Velocity.—It has been found that light and sandy soils cannot safely resist a mean velocity greater than 2 feet per second, while at the same time this velocity is sufficient to prevent plant growth and to remove silt. In firmer soils, velocities of 3 to 4 feet per second are permissible, but, except in hard pan or a material of considerable resistance, 5 feet seems to be the limiting velocity for earthen canals.

In almost any district where it is proposed to build such canals there will be some examples of ditching, on a greater or less scale, by observing which an approximate idea may be formed of the proper grade and side slopes to be given to the proposed canal. The engineer should not fail to take advantage of all such opportunities to obtain local knowledge of the district in which he is working.

47. Form and Dimensions of Cross-Section.—The cross-section of an earthen canal is customarily made in the form of a trapezoid. The side slope, or inclination of the sides to the vertical, is usually determined by the nature of the soil, and a certain depth will be found more convenient or desirable than another. By taking these and other points into consideration, the designing of a proper cross-section to satisfy the necessary requirements is simplified. The method of procedure will be best understood by an example.

Suppose that it is desired to establish the proper cross-section and grade of an earthen canal, under the following circumstances: The quantity of water to be conveyed is 250 cubic feet per second. A velocity of 2 feet per second is desired. The side slopes are to have an inclination of 1 vertical to $1\frac{1}{2}$ horizontal, and a depth of 6 feet of water is desired in the canal. What should be the form and area of the cross-section, and what the grade of the canal?

Since the velocity is to be 2 feet per second and the discharge 250 cubic feet per second, the area of cross-section must be $\frac{250}{2} = 125$ square feet.

To determine the dimensions of the cross-section, it is necessary to know the bottom width of the canal, which

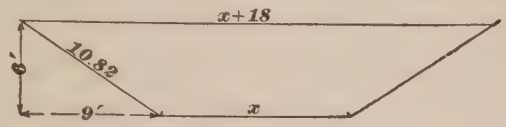


FIG. 5

is represented in Fig. 5 by x. From the data, we have $6 \times \frac{(x + x + 18)}{2} = 125$; whence, $x = 11.83$ feet.

Using a value of 12 feet, which gives the slightly greater area 126 square feet, the wetted perimeter is $10.82 + 10.82 + 12 = 33.64$ feet, and the hydraulic radius is $\frac{126}{33.64} = 3.75$ feet. Formula 2 of Art. 45 now gives

$$s = \frac{9 \times 3.75 + 35}{100{,}000}\left(\frac{2}{3.75}\right)^2 = .000196$$

This represents a grade per mile of $.000196 \times 5{,}280$, or 1.03 feet.

48. Effect of Depth on Slope.—The depth of the canal has a considerable effect on the velocity of flow. Thus, in the foregoing illustration, if a depth of 8 feet is used, all the other data remaining the same, and using the same area of 126 square feet, we shall have $8(x + 12) = 126$; $x = 3.75$.

The depth being 8 feet and the ratio of slope being 1 to $1\frac{1}{2}$, the length of the side slope is 14.42 feet, and the wetted perimeter is 32.59 feet. Therefore, the hydraulic radius is $\frac{126}{32.59} = 3.87$. Then,

$$s = \frac{9 \times 3.87 + 35}{100{,}000}\left(\frac{2}{3.87}\right)^2 = .000187$$

This represents a grade of .99 foot to the mile, as against 1.03 feet for the previous depth.

49. General Remarks on Earthen Canals.—Earthen canals, particularly in light, sandy soils, often give a great deal of trouble, even when properly side-sloped and graded, by reason of the tendency to wash; their use is, therefore, mostly confined to those very large works where the use of pipes or flumes would be out of the question. When lined with masonry, they are much more efficient, and the greater velocity that they can then safely sustain, and their consequently greatly reduced cross-section, makes their relative expense, as compared with canals having unprotected interior surfaces, less than might be imagined. Sometimes, cheap substitutes for masonry lining are employed, and it has been found in California that a good and durable lining can be made by coating the sides and bottom with a plastering $\frac{3}{4}$ inch thick, composed of one part of Portland cement and four parts of sand, the sides and bottom having previously been accurately trimmed and moistened.

50. Earthen canals are best when built entirely in excavation. It is impossible, however, to obtain this result unless the ground is exceptionally favorable and the location very carefully selected. Even then such a canal will have a greatly increased length, owing to the necessity of many deviations, in order to keep it on suitable ground. Practically, for a large proportion of their length, canals will be formed partly in excavation and partly in embankment, the material thrown out of the excavation being used, if suitable, in the embankment. Great care must be taken to carefully trim the banks to true lines.

51. The proper inclination to give to the side slopes is a point requiring very careful consideration. Both very steep and very flat slopes lead to deterioration by wash. If too steep, they fall by the undermining effect of the flow of water in the canal; and if too flat, the exposed surfaces are damaged by rain. The best guide is a careful examination of any canals or ditches that may be found already in use in the district.

When the embankment is high, it is better to keep a narrow berm between the foot of the bank and the edge of the

ditch. When very high, a center wall should be used, as for earthen dams; this, of course, adds to the expense. Fig. 6 shows a half section in which these features have been carried out.

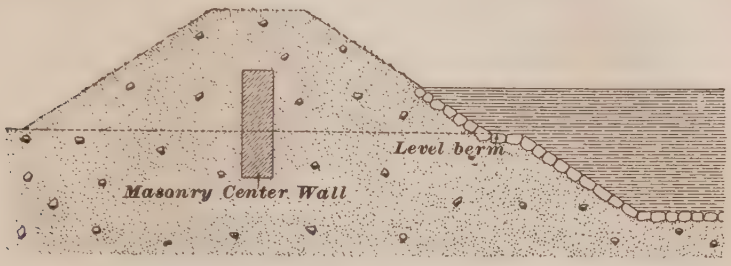


FIG. 6

52. It has been observed that in banks and excavations for both canals and railroads the effect of time and wash is always to reduce the original straight lines and sharp angles to curves and rounded edges. It would undoubtedly be an advantage to anticipate this result by giving to such work, at the start, a form somewhat similar to that which it will eventually assume. Thus, in Fig. 7, if the heavy straight

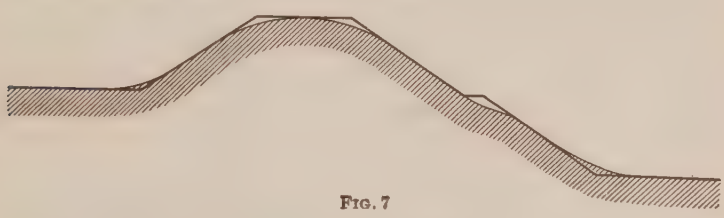


FIG. 7

lines represent the original form of the cross-section, it will gradually assume the shape shown in the shaded portions. It will be better, therefore, to favor this form in shaping the slopes of the excavation and embankment.

LINED CANALS

53. Canals Revetted With Dry Stone.—So much trouble is occasioned by the deterioration, slow or rapid, of canals with unprotected banks, and so much uncertainty

exists regarding their probable discharge, that it is often good policy and economy to line them at least with dry stone. Such an arrangement is shown in Fig. 8, which represents a canal cut in sloping ground. Any stones that can be obtained may be used for the purpose, but flat stones are preferable. It is generally best to lay the pavement continuously under the side walls, and to build these on it, as

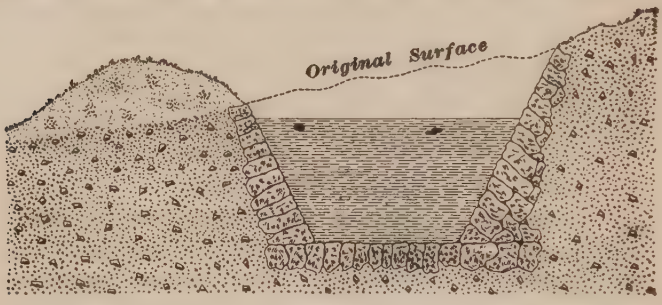


FIG. 8

shown in the figure. Some rough hammer dressing is usually required at the corners, where the side walls connect with the pavement. In laying the pavement, if flat stones can be procured, they should all be laid on edge, with a slight inclination down stream, and packed as closely as possible. All the work should be thus packed and the walls well bonded. This lining can be much improved by pointing it with cement mortar.

54. Formula for Velocity of Flow in a Canal Lined With Dry Stone.—It is very difficult to derive a formula for canals lined with stone, because the velocity of flow depends, in a large degree, on the character of the lining and the quality of workmanship. The following approximate formula may be used for a well-laid dry wall, without pointing or plastering:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{8r + 15}} \qquad (1)$$

Solving this equation for s, there results

$$s = \frac{8r + 15}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

EXAMPLE.—Referring to Fig. 8, let the bottom width of a canal lined with dry stone be 8 feet, the batter of the side walls being 1 vertical to $\frac{1}{2}$ horizontal. Let the depth of water be 8 feet, and the desired velocity 7 feet per second. What is the value of s?

SOLUTION.—Here the breadth of the waterway at the surface of the water is 16 ft. The area is, therefore, $\frac{16+8}{2} \times 8 = 96$ sq. ft. The length of the wet line on a section of the side wall, with the given batter and depth of water, is $\sqrt{4^2 + 8^2} = 8.94$ ft., and the value of p, or the wetted perimeter, is, consequently, $2 \times 8.94 + 8 = 25.88$ ft. Therefore, $r = \frac{96}{25.88} = 3.71$. Substituting known values in formula **2**, we have

$$s = \frac{8 \times 3.71 + 15}{100{,}000}\left(\frac{7}{3.71}\right)^2 = .00160. \quad \text{Ans.}$$

55. Formula for Canals Lined With Rubble Masonry.—A canal lined with masonry laid in cement constitutes a still higher type of structure. It is far more permanent in its character than the canals already considered, permits of a higher velocity of flow without injury to itself, and with a given grade and hydraulic radius offers less resistance to rapid flow.

The following formulas apply to a canal lined on the sides and bottom with good rubble masonry, pointed but not plastered:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{7.3\, r + 6}} \qquad (1)$$

$$s = \frac{7.3\, r + 6}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

56. Concrete-Lined Canals.—Now that cement is so much cheaper than formerly, the practice of lining critical parts of canals with this material is very common. This is especially true of the great canals built in the arid regions of the United States by the Reclamation Service of the Geological Survey. This kind of lining is especially adapted to porous soils, through which it would be sometimes impossible to convey water, on account of the great loss caused by percolation.

A typical section of a concrete-lined canal is shown in Fig. 9. The lining should be 6 inches in thickness; on the

outside of curves, it should be made 1 foot higher than shown in the figure. In sections where velocities exceed 12 feet per

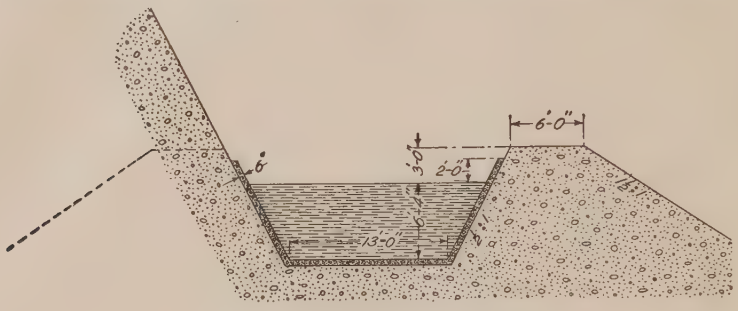


FIG. 9

second, the lining should be 7 inches in thickness. The flow in a canal of this kind can be computed by Kutter's formula, using a value of .012 for the coefficient of roughness.

EXAMPLES FOR PRACTICE

1. An earthen canal is to discharge 200 cubic feet of water per second, and the side slopes are to be 1 vertical to $1\frac{1}{2}$ horizontal. The depth of the water in the canal being 5 feet and the velocity $1\frac{1}{2}$ feet per second, what must be the value of s? Ans. .00012

2. The bottom width of an earthen canal is 8 feet, and the side slopes are 1 vertical to $1\frac{1}{2}$ horizontal. If the depth of the water is 6 feet, and the slope is 5.25 feet to the mile, what is the discharge in cubic feet per second? Ans. 432.5 cu. ft.

3. The bottom width of a canal lined with dry stone is 9 feet; the batter of the side walls is 1 vertical to 1 horizontal. The depth of the water being 9 feet, what should be the value of s, in order that the velocity may be 8 feet per second? Ans. .0015

HEAD-WORKS FOR CANALS

57. General Description.—At the point where the water from a running stream is to be diverted into a canal, a group of structures, known as **head-works,** must be built. These consist of a weir or dam built across the stream for the purpose of forcing part of the water into the canal, together with another structure, called a **regulator,** built in

the canal head, and closed by gates designed to admit the water to the canal. At that end of the weir which is adjacent to the canal is a passage, usually open, but so arranged that it may be closed, in whole or in part, by gates; this passage permits, at high water, a free flow past the regulator, and thus acts as a sluice for scouring the channel of the stream. In the canal, at a short distance below the regulator, is constructed an escape for any surplus water that may have been admitted to the canal.

58. Weirs.—Weirs may be built of brush and boulder barriers, which are easily washed away and must be replaced after each high water. They may be made of rectangular sheet piling filled with rock, or of open woodwork closed with flash boards, or of loose rock, wooden cribwork, or masonry. The form to be selected depends on the permanence of the work, the character and size of the stream, and the materials available. A favorite type of wooden weir is a series of **A**-shaped frames placed across the channel and founded on piling. The upper surface of these frames may be permanently closed with planks securely fastened, or a portion of it may be closed with planks let into grooves, and may be removed when the water rises, or for scouring. In India, a type of weir constructed in rivers of great discharge consists of long, low, flat slopes, the lower slope being 1 in 20 or more, the upper slope 1 in 2 or 3, the height being but a few feet. Along the line of the crest is built a masonry wall, and the remainder of the weir is of loose rock. Masonry weirs may be founded on crib, on piling, or on solid rock. They are given various cross-sections, similar to those of the larger dams used for storage reservoirs. The lower slope may be ogee shaped, or the water may fall into a water cushion on the lower level.

Weirs for head-works have, in some cases, been built wholly of steel, a series of steel frames being erected across the stream channel, substantially bonded into the rock surface of the bed, and covered with plate iron. Of late years, reinforced concrete has, in a measure, superseded solid

masonry for the construction of such works, this form of structure being hollow instead of solid.

59. Scouring Sluices.—The **scouring sluices,** which may be placed in the end of the weir adjacent to the canal head, consist of a series of piers between which are gates that slide vertically and are operated by screw gearing or by a similar device. Sometimes, they are closed by shutters that act automatically; these shutters fall when the water in the stream reaches the danger height to which they are adjusted.

60. Regulators.—The **regulators** at the canal heads consist of a series of sluices similar to the scouring sluices. They control the admission of water to the canal, and when closed cause it to pass down the stream over the weir. The regulator should be so placed that the water held up by the crest of the weir will pass immediately through it into the canal, and should be so close to the weir that it will practically form a part of it. Moreover, the regulator should be so inclined that the water flowing past its head will scour the bed clean at that point and prevent the deposition of silt. Regulators may differ in character according to circumstances, and may consist of wooden gates in timber framing, wooden gates in masonry or steel framing, or steel gates in masonry or steel framing. For convenience in operating the various openings in the regulator, the piers are bridged by an overhead walk from which the gates are operated. To withstand the pressure of great floods, the regulator must be firmly and substantially constructed.

CONSTRUCTIVE DETAILS

61. Overflows and Emptying Sluices.—When a large body of water is conveyed in a flume or a canal, there is always a certain degree of danger to be apprehended from a possible overflow, due to a sudden diminution of the draft without a corresponding reduction of the supply. This may be occasioned either by a shutting off of some of the outlets,

or by some accidental obstruction occurring to arrest the flow. An overflow from any cause, unless directed through proper channels, is particularly dangerous in the case of an earthen canal, and precautions must be taken to render it impossible.

It is also necessary to provide means for emptying the canal at any time, without employing the ordinary outlets used for irrigation.

The first of these requirements must be provided for, in the case of earthen canals, by building **overflows**, or **spillways**, at intervals depending on the grade of the canal, each of which should be capable of safely discharging the maximum volume of water that can reach it, supposing the entrance gates to the canal to be wide open. These spillways should be constructed precisely as described for those used in dams, taking care that they are provided with substantial wing walls and aprons, to prevent scour and wash from the escaping water. Overflows are sometimes called **escapes**.

62. The appliances used for emptying a canal consist of sliding sluice gates. These may be elaborate in construction, such as metallic gates set in masonry chambers, or they may be somewhat rudely fashioned of timber. It will be observed that these gates may have the combined office of emptying the canal and preventing overflow, but it is not safe to depend on them for the latter purpose, because, through negligence, they may fail to be opened at the proper time, whereas the action of an overflow or spillway is always automatic. Overflows and sluice gates should be located at or near the crossing of a stream, or at least at some natural depression of the ground, in order to provide for an easy escape of the water.

63. In large canals carrying a heavy body of water, both overflows and sluice gates assume considerable proportions and call for great care and skill in their design and construction. For such structures, the plans of similar works should be carefully studied, and used as guides for any particular

case. It would be impossible to give instances covering all conditions.

The most rudimentary form of sluice gate will be one inserted in a timber flume at the crossing of a stream. It will consist subtantially of a sliding gate working in a groove between upright posts. It may be operated, if small, by a lever, otherwise by a rack and pinion.

64. Culverts.—When a canal crosses a stream or the natural drainage channel at about the same level, a culvert may have to be constructed for carrying the canal over or under the channel, as the case may be. Frequently, such culverts are built of wood, usually in the form of rectangular pipes founded on piling, where the material forming the valley bottom is not stable. Proper approach wings are made at the inlet and outlet ends, and sufficient dimensions are given to carry the full volume of the canal or the flood volume of the stream. Culverts of large dimensions are usually made of masonry or reinforced concrete. The latter material has been used in some of the larger canals designed in the arid regions of the United States. For a full treatment of the subject of culvert construction, see *Culverts*.

65. Drops.—A canal usually runs through country having a greater natural fall than the canal. To compensate for the difference between the slope of the country and that which may be safely given the canal bed in order that the water in it may have the desired velocity, the canal is divided into longitudinal sections having the proper slope and connecting at the ends by **drops**, which are either vertical falls or inclined rapids. The place for a drop is determined by the point at which the canal rises above the surface of the ground.

Drops may have a clear vertical fall to a wooden or masonry apron; or the lower face of the fall may be given an ogee-shaped curve; or, to diminish the erosive action, the water may be allowed to plunge into a water cushion. Above the fall, a weir crest may be built, or the channel may be contracted, or gratings may be introduced to diminish the scour.

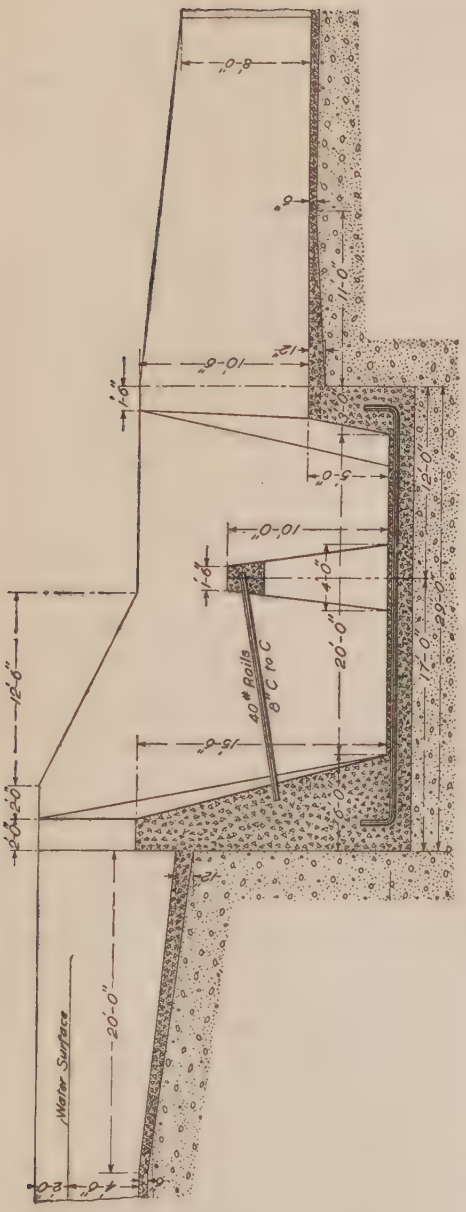


FIG. 10

66. In the past, the practice in the United States has been to build falls of wood, the bed of the stream being floored with planks and its sides protected by wings, both above and below the crest of the fall. In some cases, water cushions have been made below the crest by lining a depression in the lower channel with wood. In India and in Italy, the practice has been to construct falls of substantial masonry. The expense of such work has heretofore been considered prohibitive in the United States. Recently, in some of the drops designed by the Reclamation Service, the practice of protecting the bed and banks of drops by reinforced concrete has been adopted, and has been found to be most effective and not too expensive. An excellent example of such

a drop is that shown in longitudinal section in Fig. 10. The water from the upper canal level passes over a bed lined with 12 inches of concrete to the edge of a weir crest constructed as a masonry retaining wall 17 feet high and 2 feet wide at the top. Thence, it drops a height of 15 feet 6 inches into a water cushion 20 feet long and formed by a masonry retaining wall protecting the lower level of the canal. The height of this drop is so great that, to reduce erosive action, the force of the falling water is broken by a series of iron rails placed 8 inches apart between centers and at a height of about 7 feet above the floor of the drop. The ends of these rails are supported in some such manner as shown in the figure. The total depth of the water cushion is 5 feet, and this and the rails tend greatly to reduce the scouring action of the falling water. The concrete in the bed of the drop and in the walls lining it is heavily reinforced with iron bars running both longitudinally and transversely.

67. Turnouts.—Wherever a branch is to be diverted from a main canal, or where a canal is divided into two branches of nearly equal dimensions, it is necessary to locate a **turnout** for the diversion of the water from the main canal. These turnouts include some appliance for forcing a portion of the water from the main canal into the branch, in the head of which is an appliance for regulating the amount of water it is desired to admit. Turnouts may consist of wooden linings to the earth banks and bed of the canal, and be constructed somewhat after the fashion of a flume with wing walls properly supported by piling, etc.; or they may consist of masonry. More recently, the better forms have been made of reinforced concrete. The lining is carried up and down the canal and the branch a sufficient distance to protect the bed and bottom. In the canal are erected piers for the support of gates, or stops. These gates usually consist of planks working in grooves, with which the piers are provided, and serve to dam the water and force part of it into the branch. At the inlet to the branch, one or more gates, operated usually by wheel and gearing, are placed to admit

the proper amount of inflow. Sometimes, where but small volumes are diverted into minor distributaries, the turnout may consist of pipes or culverts laid through the banks of the main canal and controlled by appropriate gates at the inlet.

FLUMES

68. When, in order to avoid a long detour, it becomes necessary to carry the water of a canal across a valley, a **flume,** either of wood or of metal, is generally used. In the case of a city water supply, where all the installations must necessarily be on a much more permanent basis, these flumes would be replaced by masonry aqueducts, or they would, at least, be built of metal in a very substantial and perfect manner.

TIMBER FLUMES

69. General Description.—Timber flumes may be built in many ways. A simple form for a small flume, suitable for a cross-section of 4 ft. × 2 ft., is shown in Fig. 11. The dimensions of the timber are given in the figure. The bents may be 4 to 6 feet apart. In this very simple form of construction, no mortising need be used, as all the pieces can be assembled with spikes, bolts, and nails.

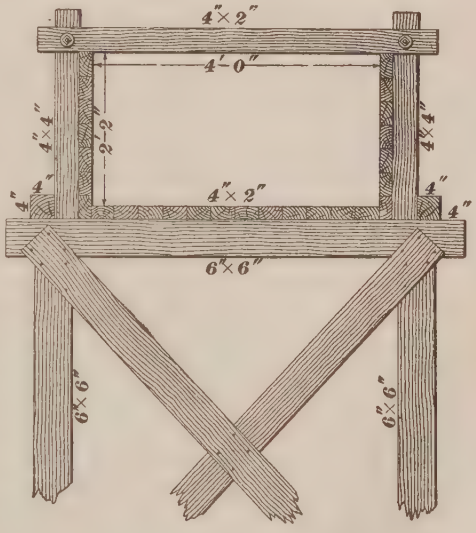


FIG. 11

An example of a larger and more perfect structure is shown in Fig. 12, which is a cross-section of the San Diego flume

in California. The inside width of this flume is 5 feet 10 inches, and the height, 3 feet 10 inches. On a hillside excavation, the flume rests on a bench 12 feet in width; while in crossing drainage lines, it rests on trestles. It is supported on mud-sills 12 in. × 2 in. laid crosswise of the bench and 4 feet apart. Longitudinal stringers of 4 in. × 6 in. timbers rest on the mud-sills, and on these are placed floor-

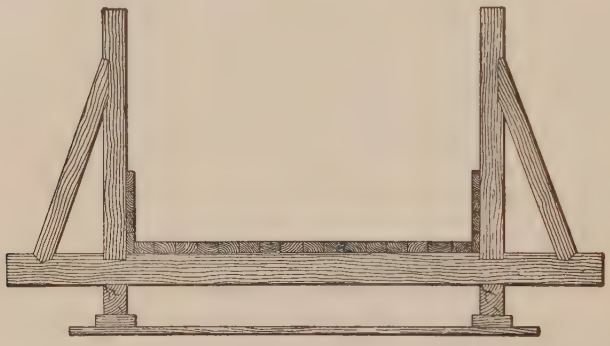


Fig. 12

beams 4 in. × 6 in., 4 feet apart. Uprights, 4 feet high, consisting of 4 in. × 4 in. scantling for supporting the sides, are let into the floorbeams, and are braced by short scantling let into the floorbeams and the posts. The interior is planked with 2-inch boards running longitudinally. Wire nails are best for spiking the planks to floorbeams, but the sides should be bolted at the joints. Nails will rot the side planking, as they are exposed alternately to air and water.

70. In the construction of wooden flumes, well-seasoned stuff should be used, and much better results, as regards flow and tightness, are obtained by having the edges and inside faces planed. A heavy coat of paint applied to the edges of well-matched planking just before spiking will make the box water-tight; without this, the joints must be calked with oakum. It is well to paint the whole of the inside, or at least the joints. When a flume is connected with an earthen canal, the greatest care must be taken to secure the point where the two connect against washing out. The flume

should enter well into the canal bank, and every possible outlet for the water should be stopped in the most effective manner.

71. Formula for the Flow of Water in Wooden Flumes.—The symbols having the same meanings as in the formulas for canals, the following formulas apply to wooden flumes:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{6.6\, r + .46}} \qquad (1)$$

$$s = \frac{6.6\, r + .46}{100{,}000} \left(\frac{v}{r}\right)^2 \qquad (2)$$

Timber flumes are generally rectangular in shape, and may be very accurately proportioned to secure the best results. The most favorable rectangular cross-section is that in which the water has a depth equal to half the width.

EXAMPLE.—A timber flume, 10 feet wide and running 5 feet deep, has an inclination of 9 inches to the mile. What is its discharge in cubic feet per second?

SOLUTION.—Here $a = 10 \times 5 = 50$, and $p = 10 + 5 + 5 = 20$; whence, $r = \frac{50}{20} = 2.5$; also, $s = \frac{.75}{5{,}280} = .000142$. Substituting in the formula,

$$v = \sqrt{\frac{100{,}000 \times 2.5^2 \times .000142}{6.6 \times 2.5 + .46}} = 2.29 \text{ ft. per sec.}$$

The discharge is found by the formula $Q = a\,v$. In this case, $Q = 50 \times 2.29 = 114.50$ cu. ft. per sec. Ans.

72. It will often be required to find the dimensions of a flume to carry a given quantity of water under fixed conditions. The exact solution of this problem is difficult, since it leads to the solution of an equation of the sixth degree; by means of a system of trial and error, however, an approximate solution may easily be obtained that will give values within the practical limits required. The following illustrative example will make the method of operation clear:

It is required to compute the dimensions of a wooden flume to convey 250 cubic feet of water per second with a grade of $8\frac{1}{2}$ feet per mile, the width of the flume to be twice the depth of the water flowing through it.

Let x equal the depth of the water in the flume; then the width will be $2x$; the wetted perimeter, $4x$; the area of the water cross-section, $2x^2$; and the hydraulic radius, $2x^2 \div 4x = \frac{1}{2}x$.

The slope is $8.5 \div 5{,}280 = .0016$; and, since the discharge is to be 250 cubic feet per second, the mean velocity v must be $250 \div 2x^2 = \frac{125}{x^2}$.

Substituting the above terms in formula **1**, Art. **71**,

$$\frac{125}{x^2} = \sqrt{\frac{100{,}000 \times \frac{x^2}{4} \times .0016}{6.6 \times \frac{x}{2} + .46}}$$

Squaring, transposing, and reducing,

$$x^6 - 1{,}289x = 179.7 \quad (1)$$

This equation is to be solved by trial. Assuming a depth of water of 5 feet for x, and substituting, we have, for the value of the left-hand member of equation (1), $5^6 - 1{,}289 \times 5 = 15{,}625 - 6{,}445 = 9{,}180$, which is much greater than the second member of the equation, and shows that the assumed value is too great.

Trying a value of $x = 4$, we have $4^6 - 1{,}289 \times 4 = 4{,}096 - 5{,}156 = -1{,}060$, which is less than the second member of equation (1), but nearer to it than the value obtained when 5 was substituted.

Trying 4.2, we have $4.2^6 - 1{,}289 \times 4.2 = 5{,}489 - 5{,}413.8 = 75.2$, which is still less than the required quantity. The value 4.3 gives $4.3^6 - 1{,}289 \times 4.3 = 6{,}321.5 - 5{,}542.7 = 778.8$, which is too great. It thus appears that a depth of water of 4.25 feet will satisfy the required condition very nearly, giving a width of flume of 8.5 feet.

IRON AND CONCRETE FLUMES

73. In the more modern works built abroad and in those built in the West by the Reclamation Service of the United States Geological Survey, steel and reinforced concrete are rapidly displacing wood as a material for the construction of

flumes. In the use of steel for long flumes, the expansion and contraction of the metal has introduced a difficulty of construction, though ordinarily of not sufficient moment to create serious trouble, since the steel usually possesses the same temperature as the water flowing through it. A steel flume is usually constructed after the pattern of a plate-girder bridge, of plate girders strengthened vertically by angles, and braced together at the top. The floor should be supported on floorbeams spaced about 5 feet apart.

74. With the introduction of reinforced concrete, structures of that material are being rapidly adopted wherever

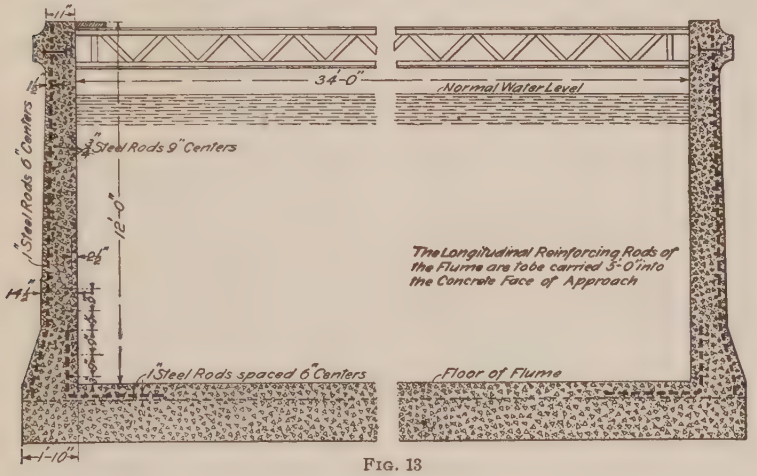

34'-0"
Normal Water Level
Steel Rods 9" Centers
Steel Rods 6" Centers
12'-0"
The Longitudinal Reinforcing Rods of the Flume are to be carried 3'-0" into the Concrete Face of Approach
Steel Rods spaced 6" Centers
Floor of Flume

FIG. 13

substantial flumes are needed. Fig. 13 shows a large flume of this kind, used on the Minidoka canal, built by the American Government in Idaho. The width of the flume is 34 feet, and the depth of water inside, 10 feet, the extreme inside depth of the aqueduct being 12 feet. The floor is of concrete, 24 inches in thickness, braced with 1-inch steel rods spaced 6 inches between centers and turned up 3 feet into the upright sides at each end. The sides taper from $14\frac{1}{2}$ inches near the floor to about 11 inches at the top, and are strengthened with two rows of $\frac{3}{4}$-inch steel rods, spaced about 9 inches on centers, running longitudinally through

the sides, and a row of 1-inch steel rods running vertically and spaced 6 inches on centers. The sides of the flume are tied together at the top by a low latticed girder.

BRIDGES AND TRESTLES FOR FLUMES

75. Bridges.—When the opening to be spanned by a flume is of considerable width, it will be necessary to use some form of trussed structure. Timber stringers should not be used for spans of more than 12 or 14 feet without trussing, unless the loads to be carried are light or the expense of trussing is much greater in proportion than the cost of the extra sizes of timber that would be required for the longer spans. In such cases, stringers with spans of 16 and even 20 feet are sometimes used. It is generally very difficult to get good, sound timber for stringers in sizes greater than 12 in. × 16 in. and 32 feet long, and even these dimensions are seldom used, owing to the expense, the difficulty of transporting such pieces, and the uncertainty in regard to their strength.

For spans between about 14 and 40 feet in length, combination king- or queen-post trusses can be conveniently used. For larger spans, plate girders, reinforced concrete structures, or any of the customary forms of trusses, as the Howe truss or the Pratt truss, may be employed.

76. Trestles.—For crossing very long depressions, trestles are as commonly employed in irrigation engineering as in railroad work. Indeed, they are more generally used in the former than in the latter, because in railroading they are frequently replaced with earthen embankments, which are to be preferred, as being more permanent for carrying trains; whereas it will rarely be found expedient to carry an irrigation flume or even a pipe line on an embankment, as the almost inevitable settling of the earth would seriously endanger the conduit. The subject of trestles is fully treated in *Trestles*, and here a few additional remarks will suffice.

When framed trestles are used, it is advisable to avoid inclined posts, mortising, and, as far as possible, different sizes of timber.

Figs. 14 and 15 represent the general features of a good system of trestling for moderate heights. The stuff used is all either 8 in. × 8 in. or 8 in. × 2 in. The 8 in. × 8 in. posts are set on the sills, which are also 8 in. × 8 in., either merely resting on the top face, or notched in ½ inch. They are held in place by **plaster plates,** of 8 in. × 2 in. stuff, bolted and spiked to posts and sills. Fig. 14 shows one of the posts and sills connected in this manner. The caps are connected with the upper ends of the posts in the same way. The posts are steadied by means of **X** bracing, of 2 in. × 8 in. stuff, as shown in the right-hand view in Fig. 15. The two pieces of the bracing are bolted together at the center, against an 8 in. × 8 in. block set between them; they are also bolted and spiked to posts, caps, and sills. These connections can be more perfectly made by first spiking the pieces together in place, and then putting in the bolts. In an emergency, all the connections may be made with spikes. The flume stringers are notched over the sills and are so disposed that joints will occur over the caps. These joints are secured by plaster plates bolted and spiked. The trestle is stiffened longitudinally by **X** bracing, of 2 in. × 8 in. stuff, as shown in the left-hand view in Fig. 15, butting under the flume stringers and against the sills, and secured laterally by plaster plates, chocks, mortising, or otherwise. The form of trestle here shown is suitable for moderate heights, say up

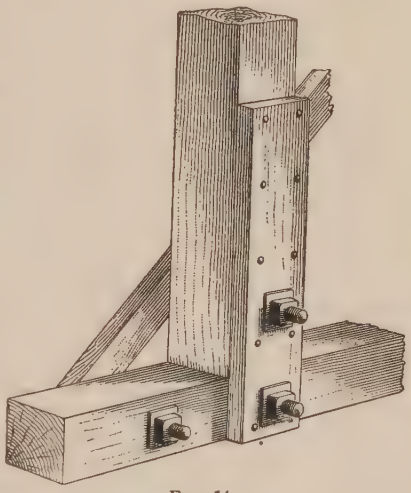


FIG. 14

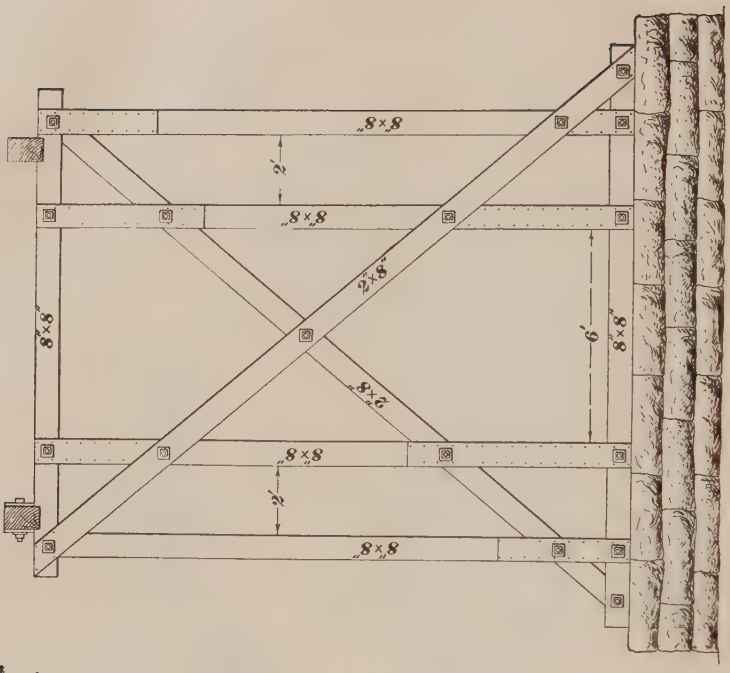
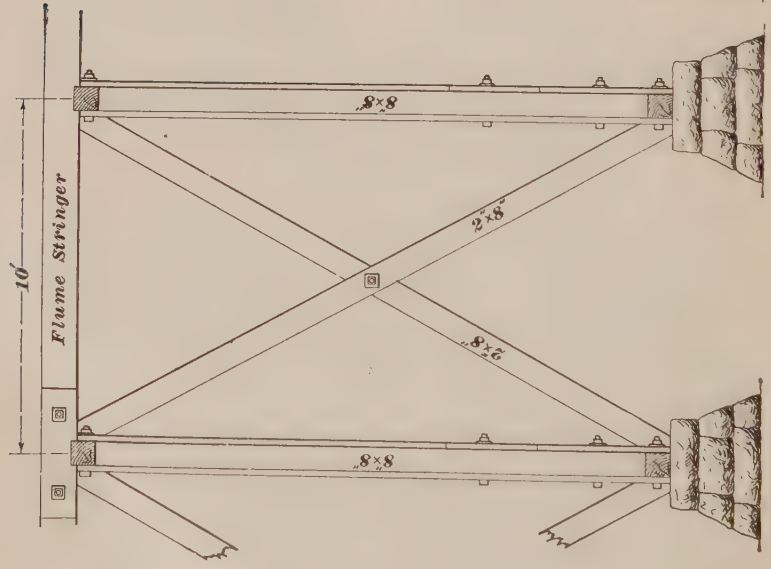


FIG. 15



to 20 feet; it will carry a 10 ft. × 15 ft. flume, and is about as light as will be perfectly satisfactory under this loading. If it were more convenient to use heavier stuff, the bents could be spaced farther apart.

PIPES AND TUNNELS

77. Pipes.—The best way to convey water for any purpose is by means of pipes. A pipe can be laid anywhere, subject to the conditions established in *Water Supply*, it is less subject to disaster and to loss of water by leakage, seepage, and evaporation, and the tendency of modern practice is to extend the use of pipes for the conveyance of water for irrigation, except where very large bodies are to be carried, when it will generally be impossible to avoid the use of canals having the dimensions of small rivers. The different kinds of pipes are fully treated in *Water Supply*.

78. Tunnels.—It frequently becomes necessary, in order to avoid long detours, to carry the line of conduit through a tunnel. If the conduit is an open canal, the tunnel will merely form an opening in the hill, affording a passage for the canal; if the conduit is a pipe line, running under pressure, the tunnel will generally form a continuation of the pipe, and will be entirely filled with water, running also under pressure. In either case, experience has abundantly shown that, except under very rare conditions, the tunnel should be lined throughout, even when it runs through rock. In modern tunneling, large quantities of high explosives are used, which greatly shatter the surrounding rock, so that fragments are continually coming away, particularly from the roof, when the tunnel is not lined, greatly interfering with the flow of water through it. Tunnels driven through earth must, of course, be thoroughly secured by lining. Such tunnels are frequently secured by timbering only, without, however, being always satisfactory. They have also, in the case of large conduits, been lined with wooden staves, as already described, the iron bands being replaced by concrete closely packed between the outside of the staves and the walls of the tunnel.

APPLICATION OF WATER TO THE GROUND

METHODS OF IRRIGATION

79. General Observations.—So far, only the collecting, storing, and transportation of water for irrigation have been considered. All these processes are merely preliminary to the great object of getting the water on the land for the purpose of producing crops; this, while the most important operation, being that up to which all the rest of the work has led, is in many respects the most complex. The problem is: Given a certain area of land, and a certain volume of water with which to irrigate it, how shall this water be evenly spread over the ground, so that every portion may receive a sufficient but not excessive degree of moisture, and that no water shall be wasted? There are many methods of applying the water; some of the principal ones will now be described.

80. Preparation of Soil.—It is essential so to prepare the soil that it will properly absorb the water applied by irrigation. The surface slope should be so fixed by grading, and the soil be put into such a condition of porosity by tilling, as to cause the water to flow over it slowly, and thus be easily absorbed. The subsoil may be opened by deep plowing, after which a thorough harrowing is necessary to place the soil in a proper condition of tilth. Successful irrigation is dependent on careful cultivation, which implies constant attention to a small area of land by each individual cultivator. Better crops may be obtained from the careful cultivation of 10 or 15 acres by a single farmer than by the careless or improper cultivation of several times that area.

81. Irrigation by Sprinkling.—Sprinkling is a method with which, when practiced on a small scale, all

are familiar. When a flower bed is to be watered, it is sprinkled by means of a watering pot. If a larger space is to be operated on, a hose with perforated nozzle, or "rose," is used, or a watering cart may be employed, delivering water in the form of a spray.

There can be no doubt that sprinkling is the best way in which water can be applied to the soil. This method fulfils all the requirements of uniform distribution, moderate and easily regulated amount, and, in consequence, permits a gradual absorption of the moisture without supersaturation of the soil or the presence of exposed surfaces of unabsorbed water, which must pass off by evaporation, thus adding to the ill effects of alkali already mentioned. The method can also be applied to rough and uneven land, without the necessity of any previous grading.

The objection to this method is the difficulty of applying it on the very large scale that is sometimes needed. It is probable, however, that by a careful and judicious system of piping it could be made more available than it is at present. It is already largely used in Florida, and is thus described by George W. Adams, of Thonotosassa: "I have a 25-horsepower horizontal boiler and a 12 in. × 7 in. × 10 in. duplex pump, with 6-inch main pipe and 3-inch laterals at the main, and running down to one and a half at extreme ends. My trees are 21 feet apart each way. I have a hydrant in the center of every 16 trees. I use the McGowan automatic sprinklers, connecting the sprinkler with hydrants by a 1-inch wire-wound rubber hose 50 feet long. I use twelve of the sprinklers at one time, and could use more just as well, each sprinkler staying in place 30 minutes, each one covering a space of from 50 to 70 feet, according to the amount of pressure given them, and discharging about 1,000 gallons. By this process I have a genuine rain, either a light one or a powerful one, at pleasure. If I wish to throw water over the tops of the trees, I use the nozzle instead of the sprinkler. I run the pump from 7 A. M. to 6 P. M. without stopping, using less than ½ cord of wood in 11 hours. I find no bad results from applying the water in the hottest sunshine, but

would if I applied it through an open hose. I think the sprinkler method of applying water requires less help than any other I have seen, and is without any danger to fruit or tree. The fireman can manage the sprinklers within reasonable distance of the pumping station. For other portions, only one man is ever needed, and it is light work for him."

In this method, it will generally be necessary to use a pump, either directly forcing the water through a hose, or else raising it into an elevated tank, from which it may be drawn as wanted.

82. Irrigation by Flooding.—Next to sprinkling, flooding is the method of applying water to the land that most naturally suggests itself. It consists essentially in spreading the water in a thin sheet over the area to be irrigated, and this may be accomplished in several ways. The first feature of this method is a ditch or canal running along the upper border or highest level of the field to be irrigated. This may be either the main conduit itself or a subsidiary ditch fed from it. It will run, with a slight fall, following the highest level, or contour line. When it is desired to irrigate the land from this ditch, one of two ways will be adopted. Small temporary obstructions, such as a few shovelfuls of earth, may be placed in the ditch, causing it to overflow at the desired points in a thin sheet. Or a break may be made in the ditch, allowing water to escape and spread over a certain portion of the field, which break will then be closed, and a new one opened, a little in advance of the first, which will irrigate an adjacent portion, and so on, opening and closing breaks with the shovel, until water has been spread with more or less regularity over the whole field. If a flume or pipe line is used instead of a ditch, sluices, permanent sliding gates, or hydrants must be placed at convenient points.

This system presupposes that the land lies on a gentle and regular slope, such as characterizes many portions of the Western American states. Almost invariably, however,

some preparatory work must be done in leveling and grading the land before this method can be employed.

The flooding system of irrigation is largely used in the cultivation of alfalfa and grass crops. It is the simplest and cheapest method, but it is very wasteful of water, and fulfils very imperfectly the requirements of a satisfactory irrigation. It distributes the water with great irregularity, overwatering some portions and scarcely moistening others. Its imperfections may be to some extent neutralized by carefully watching and guiding the progress of the water as it comes from the ditch, causing it to deviate here and there from its natural course, by a judicious use of the hoe and shovel.

There are several modifications of the general system, better adapting it to the varying topography of the ground. When a comparatively steep hillside is treated in this way, it will be better to run a series of ditches across the slope, dividing it into belts or zones, than to depend on a single ditch to irrigate the whole slope. In this case, the unabsorbed water that flows over the first belt will be caught in the ditch next below it, and passed on to the next belt, and so on.

If the ground is very level, the difficulty that then presents itself is the too rapid absorption of water in the vicinity of the outlet of the ditch before it can reach the farther limit of the field. In this case, the check-system is a useful modification.

83. In the **check-system,** a series of small ridges or checks are run across the slope, parallel or nearly so to the ditch, dividing the ground into a series of belts, in the same way as is done by the subsidiary ditches just described for hillside work. The first belt is flooded, and the water allowed to stand on it until the ground has become sufficiently saturated. The water is then drawn off by means of a break made in the ridge, and allowed to flood the next zone below, and so on.

84. Checker-Board System.—The **checker-board system** is a modification of the check-system, and is especially suited to very level land. It consists in crossing the

checks with others, more or less nearly at right angles to them, by which the whole territory to be flooded is divided into compartments. These are flooded, one or more at a time, and the water that is not absorbed is passed on to adjacent compartments.

85. The **method by furrows** is considered a very excellent one. Instead of flooding the surface generally, as just described, small ditches, or furrows, are run from the main feeder, at such an angle across the slope as to insure a gradual descent, and the water is allowed to enter these and flow through them over the field to be irrigated, the water being distributed by lateral absorption and percolation instead of by flooding the surface. As in all other systems, the course of the water should be watched and guided as far as possible, and irrigation should be followed by cultivation, or stirring and working the soil.

When orchards are irrigated in this manner, the furrows are run among and around the trees. If the land is laid out before the trees are set out, the furrows are established first, according to the most advantageous manner of suiting them to the topography of the field, and the trees are planted to suit the position of the furrows.

The ends of the furrows are connected, so that water may circulate in all directions. The water is guided by opening or closing the furrows with the shovel. Obviously, the ground must be tolerably regular in order to permit the use of this method.

86. **Subsoil Irrigation.**—This system consists in running a series of pipes, generally from 1 foot to 18 inches below the surface, very much after the manner of drain pipes. Water may be admitted into these pipes at the upper end, the lower end being closed temporarily, and allowed to escape either through perforations in the pipe or through their loose joints, if common drain pipe is used. After proper saturation, the lower ends of the pipes are opened, and they then form a drainage system for the removal of superfluous or non-absorbed water; or, the joints of the pipes

may be made tight, with openings and vertical pipes at certain intervals, from which water may flow and spread over the ground in the vicinity. This, however, constitutes rather a modification of the flooding system.

Although certain practical difficulties have been encountered in the application of this method, it gives promise of being one of the best that can be devised. It must necessarily be that which is most economical of water, and when used as first described, in the manner of drain pipes, allows the water to be drawn up by capillary action instead of sinking down by the action of gravity; loss by evaporation is thus reduced to a minimum.

87. Other Methods of Irrigating.—While there are many other systems practiced, they will be found, on examination, to consist of modifications of those described, which may be considered, therefore, as typical. The very important subject of sewage irrigation is treated in *Sewage Purification and Disposal*.

GAUGING WATER FOR USERS

88. Introductory.—When an individual owns a plant for watering his own lands, it is not necessary that the amount of water used should be measured. When the water flowing in a canal is furnished by a company or the state to more than one irrigator, some system must be adopted for measuring the amount furnished. Charges for water rates are usually based on the number of cubic feet per second delivered, and the user should know whether or not he is getting the amount of water for which he contracts.

89. Rough Methods of Gauging.—Persons whose duty it is to divide the water on behalf of a company have, as a rule, a limited knowledge of hydraulics and are capable of using but simple instruments. A very common practice on canals owned by companies is for the gate tender to judge the amount of water by its appearance. Absolute quantities are often not furnished but only proportional parts of the flow. Thus, an irrigator receives a tenth or a fifth of the

water in the ditch, rather than the number of cubic feet per second contracted for.

At the point of diversion, a wooden flume, or, in substantial works, a concrete lining, is placed in the canal, with a gate in the main canal that can be raised or lowered so as to permit or retard the passage of a portion of the water. Just above it in the side of this flume is the head admitting the water into the farmer's lateral, and this is closed by a gate consisting usually of a board that may be raised or lowered so as to admit the proper amount. Often, where water is scarce, persons in charge of the regulation of these gates, after setting them to the proper amount of opening, lock them with padlocks to prevent their being tampered with by the water users.

90. Weir Measurements.—Where greater accuracy of water distribution is desired, weirs are used. Tables showing the discharge of the weirs for certain lengths of crest and depth over the crest are furnished the men in charge of the distribution, so that they may know exactly how much water is being delivered in this manner. The practice of constructing measuring weirs in the heads of distributaries and of rating the discharge of these at various depths is rapidly being introduced into the arid regions of the United States, and the trapezoidal, or Cippoletti, measuring weir is growing in favor. The laws of Colorado and of some other states require the construction of measuring weirs, and water commissioners are employed whose business it is to ascertain the amount flowing through the various heads. The results are tabulated and furnished to the canal owners for their information.

91. Foote's Water Meter.—Several contrivances, more or less ingenious and efficient, have been devised for the purpose of readily determining the amount of water furnished to consumers. These devices are sometimes called **modules**. In order that they may meet all requirements, they should not only measure the water furnished, but also deliver a certain specified quantity.

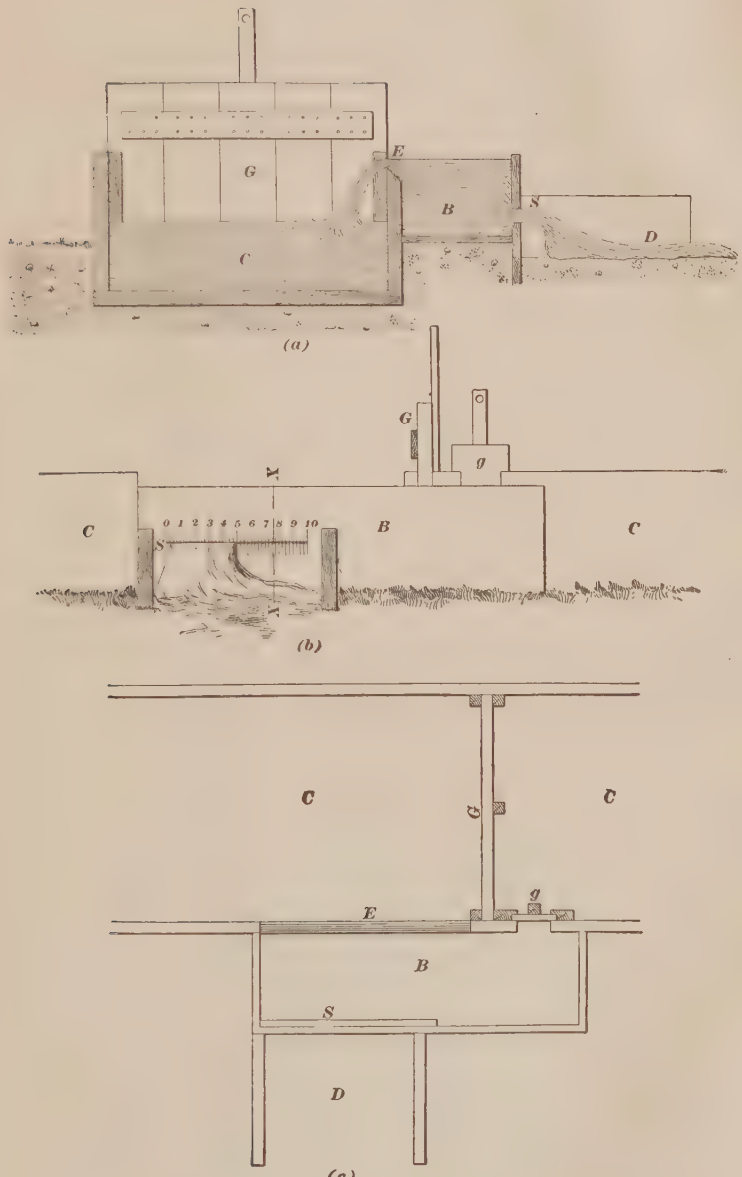


FIG. 16

Among the appliances proposed for the delivery of a specified quantity of water in a given time, one of the best is that invented by A. D. Foote, of Idaho. The general features of this meter are shown in Fig. 16.

In Fig. 16 (*a*) is represented a cross-section through the main canal or flume *C*, from which it is desired to draw a certain measured volume of water to feed the ditch *D* by means of a slot *S* in the box *B*. In order to effect this, the sliding gate *G* is partly closed, so as to impede the free flow of water in the canal *C*, and force a portion of it into the box *B*, through the small sliding gate *g*, which is partly opened for the purpose. After a few trials with the two sliding gates, their openings will be so adjusted that the proper level of the water in the box is maintained, the surplus passing over the edge *E*, and falling back again into the canal *C*, below the gate *G*.

The slot *S* is opened more or less by means of a slide on the inside of the box, the width of opening being recorded by a scale shown in Fig. 16 (*b*). This contrivance, as designed by Mr. Foote, gives the amount of water delivered in miner's inches, but the scale can be laid out so as to give the quantity in any other unit.

CROPS

92. The preceding articles cover the strictly engineering and commercial features of irrigation—all those, that is, which relate to securing the water and turning it over to the farmer, on profitable terms, for utilization. It is necessary, however, that the well-equipped engineer in this specialty should have some knowledge of how crops are raised and how the water is handled in the process.

The successful raising of crops by the aid of artificial irrigation is a scientific operation, the principles of which must be carefully studied in order to insure the best results. Long experience shows that crops do best when they receive the minimum quantity of water that they require, and, it may be added, the maximum amount of cultivation. It is found,

also, in bringing in new land, that more water is required the first year than in subsequent ones. It appears that, by a free application of water at the outset, the soil becomes gradually saturated; so that year by year the amount of water necessary for plant life diminishes, until it reaches a constant amount very much less than the original quantity required.

No fixed rules can be laid down regarding the exact amount of water required for each crop; experience, observation, and judgment are necessary to success. Some generalities, however, will be useful.

93. Alfalfa.—"Alfalfa is the greatest forage plant the world has ever known, and it should be a special crop with every irrigation farmer. It is known scientifically as *medicago sativa*, its botanical name. In the Spanish language it is called "alfalfa," while the French, Swiss, German, and Canadian people call it "lucerne." It is a leguminous perennial, and properly belongs to the pea-vine family. It is often miscalled a grass. Its term of existence has not been authentically established, but it will last the average age of a man, and instead of depleting the soil, it has a way, through its root nodules, of constantly replenishing the soil with the nitrogenous fertilizing elements of the atmosphere."—*Wilcox.*

A porous subsoil, which promotes drainage and prevents unabsorbed water from standing on the surface, is advantageous for this crop, as indeed for all others. Thorough plowing should be done in the fall, and the ground leveled before seeding in the spring. A good flooding is needed just before seeding. The seed is covered by light harrowing, or planted with a drill. It should not be buried more than 1 inch to 1½ inches deep. Too early irrigation after seeding should be avoided; it should not be practiced, as a general rule, until the plants are nearly or quite 1 foot high. When the plant has taken full possession of the ground, one good irrigation after each cutting will usually be sufficient.

"Plowing under green alfalfa as a manurial agent and soil restorative is becoming recognized in the West as a

very essential agency in preventing soil deterioration. It is, therefore, a very useful plant in following out a line of crop rotation. As a green manure or soil renovator, alfalfa is hardly equaled by any other plant. It is very rich in phosphoric acid, potash and lime, and gets a goodly portion of nitrogen from the air, leaving much of this in the soil by means of its large roots. Aside from this, when used as a green manure, there is a great deal of humus added to the soil, both by the matter turned under and by the roots. The large, long roots open the subsoil to a great depth, serving much the same purpose as the subsoil plow."

94. Wheat.—Wheat requires high land. The ground should be moist before seeding. Harrow, and plant with press drill. First irrigation may take place when the plant is 5 or 6 inches high. A second lighter irrigation may be needed about a month after the first. A third irrigation is sometimes given just as the grain is heading, if the ground has not kept sufficiently moist.

95. Oats.—Oats are treated much as wheat except that they require considerably more water. The heaviest irrigation—sometimes amounting to 1 foot in depth—is given when the plant is about 6 inches high.

96. Rye.—Rye is the easiest grown of all the cereals, and needs the least water; sometimes only one light watering is sufficient.

97. Corn.—Corn requires a great deal of preparation of the soil, and of cultivation after planting. Excessive irrigation must be avoided. One or two irrigations will be sufficient. A watering will generally be wanted when the tassels are formed. Altogether, this crop requires a good deal of attention.

98. Grasses.—Much that has been said of alfalfa applies to the grass crops. One general rule for hay crops is not to irrigate for a considerable time previous to cutting, so as to permit a thorough assimilation of plant food, and to allow the ground to acquire a proper condition for cutting and curing the hay.

AMERICAN LAWS ON IRRIGATION

99. Priority of Appropriation, and Prorating.—The questions of riparian rights and the right of priority of ownership of water have assumed a new aspect under changed conditions. This has not yet taken definite shape, and there is no set code regulating the rights of water for use in irrigation. The old law of **riparian rights** can be enforced only for protection of water that has been put to a beneficial use. The right of **priority of appropriation** was maintained for a long time in some of the states of the West, but that also has practically been superseded by the principle of **beneficial use.** Instead of the ownership of the water going with the land, as under the older common law, the flowing water is not classed as property that can be owned by any person. Where the land originally belonged to the United States, the unused waters both above and beneath the land still belong to the government. The use of the water is guaranteed to appropriators to the extent to which they put it to beneficial use, and generally in the order of priority in which they make such use. While the right of appropriation was originally based on the need of water and the right to take it, changed conditions have resulted in its modification by requiring evidences of beneficial use.

A still later development has been the system known as **prorating.** This consists in dividing water proportionally to the amount available. No consideration is given the user nearest the ditch head, who may have been the first irrigator. He receives the same proportion of his usual share as do his associates. In time of scarcity of water, the application of priorities must give way to proportional division of the supply available. It is not possible to deprive a large number of the water users of their supply and cause their crops to be destroyed, in order that the full proportion may be given the favored few prior owners. Accordingly, there is a tendency

to abandon the strict observance of priorities in favor of prorating water. The practice of judicial decision seems to be in favor of the view of changing the use of the water to which an irrigator is entitled from one piece of land to another, but this right could only be held to be pertinent to some specified piece of land, and not to land in general, or wherever the irrigator might desire to use it. In other words, his right to water would not be owned separate and apart from the land.

These same questions become still more complicated when considered in connection with property in water owned by a canal company. It is then necessary to recognize the difference between rights to divert water, to carry it, and to furnish water to users and to charge for it. Such rights are distinct from those bearing on the actual use of water in irrigating the land, and are considered to be enjoyed by a canal company as a common carrier. The company has no actual ownership in the water in the sense that it owns the canal and the regulating works; its position is that of a trustee conveying the water to those who will put it to actual beneficial use.

100. Reclamation Law.—The President of the United States approved June 17, 1902, an Act known as the **Reclamation Law.** This created a means whereby the proceeds of the sales of public lands in thirteen arid states will become a revolving fund held in the Treasury of the United States, and known as the **Reclamation Fund,** to be used in the examination, survey, construction, and maintenance of irrigation works for the storage, diversion, and development of waters for the reclamation of the lands in those states and territories. The law further provided that the reclaimed lands should ultimately be sold to bona-fide settlers at a uniform price per acre that would be sufficient to reimburse the government for the entire outlay, such proceeds becoming again available for further reclamation. In accordance with this law, and to carry out its provisions, a branch of the United States Geological Survey, known as the Reclamation

Service, was established. The Service examines the lands, determines their possible capability of reclamation, recommends their withdrawal from sale or preemption under other laws, prepares plans for projects for their reclamation, and, if the latter are approved by the Secretary of the Interior, undertakes the construction, and, if necessary, the subsequent maintenance of the works.